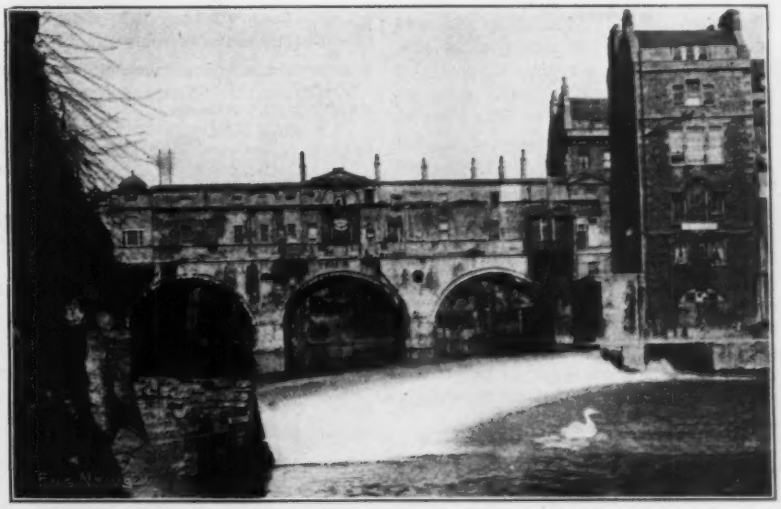
Engineering News

Vol. 67, No. 19. 505 Pearl Street, New York May 9, 1912

An English Bridge Surmounted by Buildings

One of the interesting old bridges still remaining in use in England is the Pulteney Bridge, in the city of Bath, carrying Pulteney St. over the River Avon. Its most curious feature is that it has along each side of the roadway a row of two-story buildings. A view of the bridge is given in the accompanying cut, which shows also the weir for an old water-power mill. The bridge was constructed in 1770 by Robert Adams, architect, for Sir William Johnstone Pulteney, the owner of the Bathwick Estate, and was built in connection with the laying out of Pulteney St. and other thoroughfares. The city repairs and maintains the street surface, but the substructure belongs to the estate. For the photograph and other information we are indebted to Charles R. Fortune, City Surveyor of Bath.

The bridge has three segmental arch spans of about 33 ft. 6 in., with piers 8 ft. thick. The height from water level is about 21 ft. to the crown of the arch and 26 ft. to the roadway. The width is about 58 ft. over the spandrel walls, with 34 ft. between the two rows of two-story buildings. The buildings on the south side are 10 ft. wide or deep, while those on the north side are 23 ft. 6 in. deep, having the outer 10 ft. cantilevered beyond the face of the bridge. The bridge is constructed of Bath stone (a yellow sandstone which blackens with age), upon a foundation which presumably is in the lias clay. One of the piers gave way in 1803, damaging the houses and necessitating the rebuilding of one side. There are signs of settlement in the piers and abutments.



THE PULTENEY BRIDGE (WITH BUILDINGS) OVER THE RIVER AVON AT BATH, ENGLAND

A Newspaper Story and an Engineer's Account of a Dam "Failure"

The difference between a newspaper account of a dam "failure" and what actually happened to the dam is seldom greater than in the following case, which is respectfully referred to the heads of the new School of Journalism at Columbia University. A press dispatch dated Lebanon, Penn., Apr. 9, read as follows:

> Held back only by the thin inner wall of the dam, 60,000,000 gal. of water in a storage reservoir threatened this valley today after the breast of the dam had caved in. The reservoir was drained.
>
> This was the largest of the city's three storage basins. The fact that the inner wall held after the masonry had fallen is considered to be almost miraculous.

What occurred has kindly been reported at our request by T. R. Crowell, city engineer of Lebanon, as follows:

Dam No. 2, as it is called, of our gravity water supply is an earth dam. Originally the breast, built in 1884, was 25 ft. high and about 400 ft. long at the flood line. In 1910 the breast was raised 17 ft., with a resulting length of about 720 ft. The inner slopes are 2½ to 1 for a vertical height of 25 ft. and 2 to 1 for 17 ft. The outer slopes are 2 to 1. The material in the embankment is approximately 50,000 cu.yd.

On Monday, Apr. 8, a slip occurred, confined entirely to the outer slope of the breast. The material dislodged was only some 140 cu.yd.

Before the slip happened there had been a heavy downpour of rain. Frost had just disappeared. The reservoir was not drained. In response to a telephone message from the keeper of the dams, orders were given to lower the water. A visit made to the dams by the water superintendent and the writer showed that there was no necessity whatever of draining the reservoir. The gates were therefore closed after the water had been lowered about 5 ft.

On Monday, Apr. 15, the dam was inspected by engineers from the State Department of Health and State Water Supply Commission, who gave out for publication the statement that the occurrence was trivial in its nature and that neither life nor property was endangered thereby.

The press dispatch which you give reminds one of the classic story of Cuvier's reply to the French Academicians, when they submitted their definition of a crab.

The dam does not hold 60,000,000 gal. Any water escaping therefrom, could not possibly reach the valley in which Lebanon lies, but would find its way eventually to the Conestoga Valley, of Lancaster County. The reservoir was not drained. There was no masonry to fall. In other respects the story is substantially true.

The Strength of Rolled Zinc*

By Herbert F. Moore†

Zinc, either rolled or cast, has no well defined yield-point and its elastic limit is very low. The ultimate tensile strength of thin rolled zinc plate (not more than 0.05 in. thick) is about 24,000 lb. per sq.in. The modulus of elasticity of zinc in tension is about 11,500,000 lb. per sq.in.

The stress per sq.in. of area sheared, developed in punching or shearing rolled zinc plates, is about 40% of the stress developed in punching or shearing mild-steel plates.

The energy per sq.in. of area sheared, per in. thickness of plate, required to punch or shear rolled zinc plates is about 30% of the energy required to punch or shear mild-steel plates.

The ductility of rolled zinc is much less than that of mild steel and the ductility of zinc plates with the grain is greater than the ductility across the grain.

*Abstract of Bulletin No. 52 of the University of Illinois.

†Assistant Professor in theoretical and applied mechanics, University of Illinois.

The occasional use of sheet-zinc hangers for supporting electric cables called the attention of the Matthiessen & Hegeler Zinc Co., of La Salle, Ill., to the question of the strength of rolled sheet zinc, and it was found that few data were available. The matter was referred to the attention of the engineering experiment station of the University of Illinois, and the general subject of the strength of rolled zinc seemed to be of sufficient importance to warrant making tests. A series of tests of strength of zinc was made in the Laboratory of Applied Mechanics of the university by the writer under the general direction of Prof. Arthur N. Talbot.

In addition to tension tests of zinc, tests of rolled zinc under punching and shearing were made, and in the course of the investigation, several tests of cast zinc were made including torsion and compression tests, also cold-bending tests of rolled zinc. [The results of all these tests are recorded in full in the *Bulletin*. —ED.]

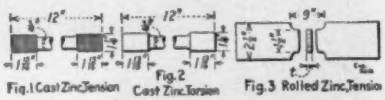
The zinc tested was obtained from the zinc works of the Matthiessen & Hegeler Zinc Co., of La Salle, Ill., and was smelted from ores from the Joplin, Mo., district. Several specimens of cast zinc were furnished which were remelted spelter, poured directly into molds. There were 18 sheets of rolled zinc, each 18 in. by 20 in., varying in thickness from 0.006 in. to 1 in. Three sheets of each thickness were furnished and each of the three sheets was from a different heat. No special precautions as to heat treatment were taken either with the cast zinc or with the rolled zinc. As a check on the values obtained for the zinc plate furnished by the Matthiessen & Hegeler Zinc Co., tests were made on sheet zinc purchased in the local market at Champaign, Ill.

Test Specimens

The specimens for tension tests of cast zinc were similar in form and dimensions to the specimen shown in Fig. 1. The cast zinc was furnished in bars 1¼ in. square by 12 in. long and machined to the size shown. Fig. 2 shows the form and dimensions of the specimen of cast zinc tested in torsion. The specimens of cast zinc tested in compression were circular cylinders 1 in. in diameter by 1½ in. long. They were machined all over.

The specimens for tension tests of rolled zinc had the form and dimensions shown in Fig. 3. The strips from which they were formed were sheared from the 18-in. by 20-in. plates in all cases except some of the plates 1 in. thick from which it was found impossible to shear strips without seriously injuring the plate. In the case of these thick plates, the strips were cut from the plate by drilling. The specimens were then machined on a shaper to the form shown in Fig. 3.

The punching and shearing tests for rolled zinc were made on that portion of the 18-in. by 20-in. plates which remained after the tension-test pieces had been cut out. The holes punched in the punching test were separated by a distance at least 1½ times the diameter of the punch. The shearing tests were made on specimens cut from the portion of the plate left after the punching test had been made. The shearing-test specimens were cut 1 in. wide and were sheared in double shear. Some specimens were sheared across the grain (perpendicular to the direction of rolling) and some with the grain (parallel to the direction of rolling).



FIGS. 1, 2 AND 3.—TEST SPECIMENS

The sheet of zinc bought in the local market differed in size from those furnished by the Matthiessen & Hegeler Co., but the general method of cutting specimens from it was the same as that just described.

Tension Tests

Table I shows the summarized result of the tension tests of zinc. There was considerable variation in strength shown by individual specimens of cast zinc, the ultimate strength ranging from 6050 lb. per sq.in. to 12,220 lb. per sq.in. In the case of the rolled zinc, however, there was little variation in strength shown by the individual specimens. The extreme values were in general within 10% of the mean. No well defined yield point could be determined in any tension test.

Punching and Shearing Tests

The punching tests of rolled zinc were made with hardened-steel punches and dies mounted on the weighing table of a testing machine. The punches used were 0.505 in., 0.747 in., and 1.001 in. in diameter, respectively, and the corresponding dies were 0.61 in., 0.86 in., and 1.13 in. in diameter, respectively. The punches were flat faced.

Most of the punching tests were made with a speed of punch so slow (0.1 in. per minute) that the weighing beam of the testing machine could be kept in balance by hand as the test progressed. It was desired to run some of the punching tests at a higher speed, but as it was impossible to keep the weighing beam accurately balanced by hand in these tests, a steam-engine indicator was attached to the testing-machine weighing beam, so that the compression of the indicator's spring measured the load on the punching tool.* In the punching tests of very thin plate only the maximum load was recorded.

*For description of this apparatus see the Proceedings of the American Society for Testing Materials for 1908, page 653.

TABLE I. TENSION TESTS OF ROLLED ZINC

Thickness of plate, in.	Number of specimens*	Ultimate stress, lb. per sq in.	Modulus of elasticity, lb. per sq.in.	Elongation in 8 in., per cent.	Reduction of Area, per cent.	Direction of testing
1	4	21,340	10,450,000	4.85	6.58	With grain
1	3	23,950	10,370,000	0.31	5.37	Across grain
0.10	6	22,280	10,650,000	18.75	....	With grain
1.10	6	33,620	13,700,000	14.3	....	Across grain
0.006	6	26,480	14,800,000	6.5	....	With grain
0.006	3	33,830	.....	.....	....	Across grain

*The values given are the average results.

Shearing tests of rolled zinc were made on a 100,000-lb. Riehle testing machine fitted with autographic recording apparatus. A hardened-steel shearing tool cut the specimen in double shear. Fig. 4, shows the arrangement of the shearing tool. Most of the shearing tests

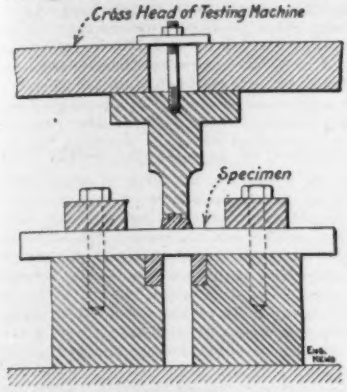


FIG. 4. APPARATUS FOR SHEARING TEST

were run at a low speed of tool (0.01 in. per minute) and the weighing beam of the testing machine was kept balanced by hand. Some shearing tests were made at a high speed, and for these the steam-engine indicator device for weighing loads was used as in the punching test at high speed.

The data for the punching and shearing tests were all autographic except for punching tests in very thin plate. Fig. 5 shows diagrams for punching tests, and Fig. 6 shows typical diagrams for shearing tests of zinc plate. In Fig. 6 are also shown diagrams of shearing tests of steel plate for comparison.

TORSION AND COMPRESSION TESTS

Torsion tests were made on six test pieces of cast zinc. The results of these tests are summarized as follows: Maximum fiber stress at limit of proportionality of stress to angle of twist, 5450 lb. per sq.in.; computed maximum fiber stress at rupture, 15,260 lb. per sq.in.; modulus of elasticity in shear (torsion), 4,570,000 lb. per sq.in. The character of the fracture was similar in form to that of cast iron under torsion.

Compression tests were made on four short cylinders of cast zinc. The results of these tests were as follows: Fiber stress at limit of proportionality of stress to compression, 1620 lb. per sq.in.; modulus of elasticity, 6,900,000 lb. per sq.in. The behavior of cast zinc under compression is worthy of note. Judged by fractures in tension and in torsion tests, cast zinc is a brittle material and under compression a shattering failure might be expected—such a failure as cast iron exhibits in compression. What actually happened to the compression-test pieces of cast zinc was a gradual flattening out, such as occurs with soft steel. No maximum load could be determined. Cast zinc is evidently a plastic but not a ductile metal.

DISCUSSION OF RESULTS

An examination of the results of the tension tests shows that zinc, either cast or rolled, is imperfectly elastic under very low stresses. The results of Meyer's test are in agreement with the results of the Illinois tests as regards this general conclusion, though in the Illinois tests permanent set was detected at lower stresses than in Meyer's tests. Whether the elastic limit be defined as the lowest stress under which a material is given a permanent set, or as the lowest stress at which Hooke's law is found to be inexact, the elastic limit is very low and very poorly defined. Meyer reported a yield-point of zinc in the tables of results of his test, but in the accompanying discussion he stated that the yield-point was poorly defined. In the Illinois test no well defined yield-point could be detected, and none was reported.

The ultimate tensile strength of cast zinc depends on the temperature of pouring and other factors, and varies between wide limits. Thin plates of rolled zinc are

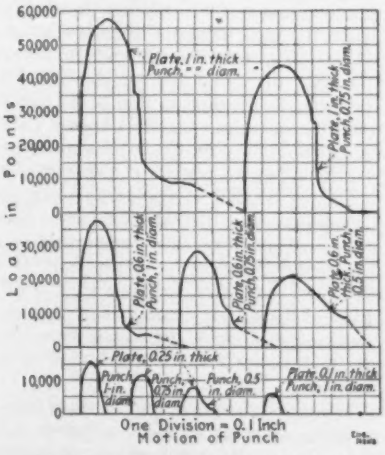

Load in Pounds
60,000
50,000
40,000
30,000
20,000
10,000
0
Plate, 1 in. thick Punch, 1" diam.
Plate, 1 in. thick Punch, 0.75 in. diam.
Plate, 0.6 in. thick Punch, 1 in. diam.
Plate, 0.6 in. thick Punch, 0.75 in. diam.
Plate, 0.6 in. thick Punch, 0.5 in. diam.
Plate, 0.25 in. thick
Punch, 1 in. diam.
Punch, 0.75 in. diam.
Punch, 0.5 in. diam.
Plate, 0.1 in. thick Punch, 1 in. diam.
One Division = 0.1 Inch
Motion of Punch

FIG. 5. PUNCHING TEST DIAGRAMS; ROLLED ZINC

relatively stronger under tension than thick plates. For plates under 0.05 in. thickness, 24,000 lb. per sq.in. would seem to be about the value to be used for the ultimate tensile strength of rolled zinc. For plates over 0.05 in. thick, 21,000 lb. per sq.in. would seem a reasonable value to use. Rolled zinc is somewhat stronger in tension across the grain than with the grain.

A higher tensile strength of rolled zinc was obtained by increasing the rapidity of application of load. Rapidity of stressing may in part account for the fact that higher values of tensile strength were found by Meyer and by the Matthiessen & Hegeler Co. than were found at Illinois. The speed used in the Illinois tests was lower than is sometimes used in commercial testing.

Shearing Strength—The values of shearing strength of rolled zinc reported in this bulletin were determined with the purpose of throwing some light on the problem of what sizes of punches and shears should be used in working with zinc plates. Two factors which are important in their influence on the design of punches and shears are the maximum force to be exerted during the process of punching or shearing and the energy required to complete the punching or shearing. The maximum force to be exerted determines the strength of frame, ram, gearing and other parts of the punch or shear; the energy required determines in a large measure the weight of the flywheel, size of belt and power required for power-driven punches and shears, or the power and amount of water or air required for hydraulic or pneumatic punches or shears.

The maximum force to be exerted during the punching or shearing action is in general proportional to the area actually sheared; the mean force during the action is approximately proportional to the maximum force and consequently to the area under the action of the punch or shear. This may be seen from the general similarity of shape of the punching and shearing diagram for different thicknesses of plate in Figs. 5 and 6. The energy required to punch or shear any plate will be approximately proportional to the area to be sheared by the punching or shearing tool multiplied by the distance traveled by the tool during the punching or shearing action, that is by the thickness of the plate.

The results of the punching and shearing tests of zinc plate show an average value of shearing stress developed of 19,400 lb. per sq.in. for the punching tests and of 17,100 lb. per sq.in. for the shearing tests. The average value of the energy required was 8110 lb. per sq.in.

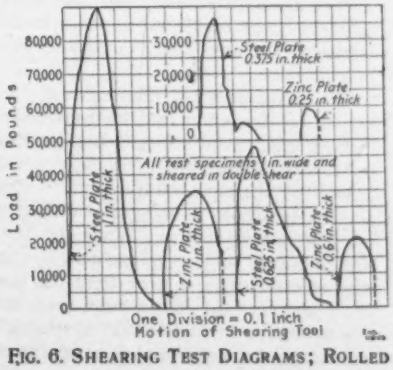


FIG. 6. SHEARING TEST DIAGRAMS; ROLLED ZINC AND MILD STEEL

per inch thickness for the punching tests, and 4850 in.lb. per sq.in. per inch thickness for the shearing tests. Evidently the frictional resistance of the metal pushed out (the "wad") is greater in punching than in shearing, as is shown by the slightly greater stress developed

and the markedly greater energy required.

The results of the punching and shearing tests of mild-steel plates tested for purposes of comparison with the rolled zinc show the following average values: Shearing stress developed, 50,040 lb. per sq.in. for the punching test, and 43,270 lb. per sq.in. for the shearing test; energy required, 25,200 in.lb. per sq.in. per inch thickness for the punching test and 16,910 in.lb. per sq.in. for the shearing test. These results do not differ widely from those of other tests of mild steel in punching and shearing.

An examination of the results of the punching tests shows that the larger the punch for any given thickness of plate, the less the unit energy required to punch the plate. In both punching and shearing tests, the maximum stress developed was slightly increased under increased speed of punching or shearing tool. Both punching and shearing tests were made with flat-faced tools, the object being to bring the stress as uniformly as possible on all parts of the area under shear. By the use of beveled punching and shearing tools, the maximum force resisting shear would have been reduced.

Ductility and Elasticity—In the series of tests described in this bulletin the elongation and the reduction of area after rupture in tension and the results of the cold-bending tests were all used to throw light on the ductility of the zinc. Unfortunately, there is no well defined quantitative standard for the comparison of the ductility of different metals. It is evident, however, from these results that zinc is much less ductile than wrought iron or mild steel and that it is less ductile across the grain than with the grain. For a zinc plate which is to be stamped or bent into shape (for example in the making of zinc elements for dry batteries) a severe cold-bending test would seem to be of considerable value in determining the acceptability of a shipment of zinc plate.

The researches of Martens and of Meyer on the effect of the heat treatment of zinc on its strength and ductility indicate the desirability of measuring and controlling the temperature in the rolling process and show the danger of rolling at too high a temperature. While the ductility of zinc is low as compared with that of steel, it is evident from the low and poorly defined elastic limit, from the loss of energy in "mechanical hysteresis" and from the behavior of compression test pieces that the zinc possesses a relatively high degree of elasticity.

Record for Culebra Cut Excavation—The highest daily record of excavation in the Culebra Cut proper, Panama Canal, was made on Apr. 11, when 44 steam shovels excavated 68,505 cu.yd. in a working day of eight hours. This is an average of 1550 cu.yd. per shovel.

United States Government Tests on Reinforced-Concrete Beams

The Structural Materials Laboratory of the United States Geological Survey was founded at St. Louis, Mo., in 1904, in connection with the world's fair being held there at that time. At this laboratory there was carried on for six years extensive tests of various structural materials, including a number upon various kinds of concrete and reinforced-concrete specimens. The scheme of the laboratory in these latter tests was to proceed from the simplest forms of cement, mortar and concrete through concrete tension and compression tests to various reinforced-concrete beams and columns. In July, 1910, by act of Congress the laboratory was transferred from St. Louis to Pittsburg and from the direction of the Geological Survey to the United States Bureau of Standards, under whose auspices further work will be carried on. Preliminary reports of the tests on concrete materials include Bulletin No. 331 of the U. S. Geological Survey, which describes the tests of portland-cement mortars and their constituent materials, and Bulletin No. 344 of the U. S. Geological Survey, which covers the tests of plain concrete beams. The next step in the testing, that is the test of plain reinforced-concrete beams, was completed at the St. Louis laboratory, but the data for the test were transferred to the U. S. Bureau of Standards when the laboratory was transferred and have been just published by that bureau as a "technologic paper."* As the book is a 200-page large-size octavo with a multitude of tables and diagrams it is quite impossible to even abstract it here, but we give below a short description of its contents.

The tests comprise simultaneous compression tests on 8x16-in. cylinders and 6-in. cubes and simply reinforced beams with varying percentages of reinforcement. The series consisted of 336 beams (three of which were not tested) together with the corresponding cylindrical and cubical test pieces for compression tests and 48 bond test pieces to determine the adhesive strength of the aggregates used. Representative material of four classes of aggregate, gravel, crushed granite, crushed limestone and cinders, were used in making up the test pieces which were tested in triplicate at the ages of 4, 13, 26 and 52 weeks, respectively. The report says:

> The report of these tests has been confined exclusively to a presentation of test data in such a way as to indicate with considerable certainty the various elements which affect the design of architectural and engineering structures. Studies have, therefore, been made near maximum load between external load conditions, as measured by the bending moment, and resistance of the beam as determined by the resisting moment. There is included the relation between the unit stress at maximum load and the yield point of the reinforcement.
>
> Studies have been made for maximum load and for unit stresses of 16,000 and 32,000 lb. of the relation between the effective percentage of reinforcement and (1) the resistance of the beam as measured by $\frac{M}{bd^2}$, (2) percentage depth to the neutral axis, (3) unit compressive stress in the extreme upper fiber, (4) the deflection at the center of the beam. The results are presented diagrammatically and in greater detail in tabular form. The change of such relation with age and the difference due to the nature of the aggregate have also been determined and are presented.
>
> The development and spacing of the tension cracks below the neutral axis for the various aggregates, ages, and percentages of reinforcement is plainly shown by photographs of the beams.
>
> There is also included a study of plain concrete in direct compression. The growth in strength with age and the increase in stiffness, as measured by the initial modulus of elasticity, are clearly brought out. The variation with age of the ratio of the yield point to the ultimate strength for the different aggregates has been determined.
>
> No extended discussion of the theory of reinforced concrete is attempted in this bulletin, and theoretic formulas have been introduced only where necessary for the intelligent presentation of the data. The formulas used are, however, not original with the bulletin, but are based on assumption which are thought to approximate the truth.

The preliminary description includes accounts of the methods of tests of the cement, the sand, the aggregates, the reinforcement and the separate test pieces. Following that is a chapter on the result of tests, which discusses the tables which include all the data of the tests and in addition takes up the analysis of the beam at the first crack, the analysis at maximum load and the effect of variations in the percentage of reinforcement. Finally, in this portion is a discussion of the compression tests, taking up the influence of age on relative strength of cubes and cylinders and upon the modulus of elasticity and yield point and the ratio of the yield point to the ultimate strength.

The variation in the constants entering into the composition of the beam and the completeness of the record of the tests are such that a great number of tables and diagrams are needed to present the results. At the same time, these results are most comprehensive and permit of a thorough study of the behavior of simple reinforced-concrete beams.

Conclusions

As a result of the tests, the authors have made the following conclusions:

*Technologic papers of the Bureau of Standards, F. W. Stratton, Director. No. 2, The Strength of Reinforced-Concrete Beams; Results of Tests of 333 Beams (First series), by Richard L. Humphrey and Louis H. Losse (June 27, 1911). Washington, Government Printing Office, 1912.

REINFORCED CONCRETE IN FLEXURE—The unit elongation of the extreme lower fiber at the time of the occurrence of the first crack is 0.00012, for stone and gravel concrete, 0.00018 for cinder concrete, and equals the unit elongation of the lower fiber at rupture of a beam of the same concrete without reinforcement.

The unit elongation of the extreme lower fiber at time of the occurrence of the first visible crack increases with the percentage of reinforcement.

The distance between cracks due to tension varies with the percentage of the reinforcement, slightly with the kind of aggregate, and is independent of the age of the concrete.

The ratio of the unit stress in the reinforcement at maximum load to the yield point varies with the percentage of the reinforcement, is independent of the age of the concrete, and shows no regular variation with the initial modulus of elasticity of the concrete.

The unit bond stress at a load just below maximum, for a unit stress in the reinforcement somewhat less than the yield point, is almost constant and equal to about 120 lb. per sq.in. It increases slightly as the percentage of the reinforcement increases, increases somewhat with age, and with the strength of the concrete.

Basing the resisting moment of a reinforced-concrete beam on the tensile stress in the reinforcement, and neglecting the tensile resistance of the concrete, the ratio of the resisting moment to the bending moment is always less than one between 0.5% and 1.7% "effective" reinforcement. This ratio is independent of the kind of aggregate used, decreases with age, and increases with the percentage of reinforcement.

The behavior of a reinforced-concrete beam, having the reinforcement in two horizontal planes is identical with a beam having all the reinforcement concentrated in one plane, and equal in amount to the reinforcement in the lower plane plus that in the upper plane multiplied by the ratio of the distances of the upper and lower planes from the neutral axis.

The percentage depth of the neutral axis below the top of the beam increases with the percentage of reinforcement and increases or decreases with different amounts of reinforcement as the intensity of loading increases, depending upon the ultimate strength of the concrete as influencing a compression failure, and the relative stiffness of the concrete for varying degrees of stress as determined by the character of the stress deformation diagram of the cylinders (fixed by the proportions, consistency, kind and age of concrete).

For all practical purposes the percentage depth to the neutral axis for a given unit stress in the reinforcement may be taken as independent of the age of the concrete.

The character of the concrete as measured by its modulus of elasticity exerts an influence on the position of the neutral axis.

For any given percentage of effective reinforcement the moment of resistance varies as the ordinates of a straight line, and based on the moment of resistance of a plain concrete beam the moment of resistance for a reinforced-concrete beam is proportional to the effective percentage of reinforcement.

The moment of resistance is appreciably affected by the character of the aggregate used and the age of the concrete.

For a given unit stress in the reinforcement the deflection increases with the percentage of "effective" reinforcement and increases as the modulus of elasticity and ultimate strength of the concrete decreases.

The deflection of a reinforced-concrete beam increases in direct proportion to the increase in the unit stress in the reinforcement.

The unit compressive stress in the extreme upper fiber varies with the percentage of effective reinforcement as the ordinates to a straight line.

Excepting cinder concrete for the earlier ages and low percentages of effective reinforcement, the unit stress of 16,000 lb. per sq.in. in the reinforcement corresponds to a unit compressive stress in the extreme upper fiber of at least 1000 lb. per sq.in.

CONCRETE IN COMPRESSION—The conclusions as to the general effect of age on the various elements (ultimate strength, initial modulus of elasticity, and yield point) that determine the value of a given concrete are probably of fairly general application.

It should be recognized that the results obtained for each class of material —i.e., granites, limestones, gravels, etc. —used in these tests may not be applicable to other materials of the same class obtained from a different source. There are too many elements affecting the strength of concrete to warrant the assertion that because an individual of a class—i.e., granite—gives excellent results, that, therefore, all granites will give equally good results. The gravel used in these tests had smooth round surfaces but developed a compressive strength but little inferior to that of the granite, and appreciably greater than that of the limestone. Yet it cannot be stated that the use of any given gravel for concrete is permissible, because this particular sample, which originated from a hard, flinty rock and is well graded, gives excellent results. A gravel deposit will necessarily partake of the physical properties of the rock from which it is derived and no good results can be hoped for by the use of a gravel which originated from a soft, chalky limestone or from some weak, cleavable shale or sandstone. The compressive strength of the stone affects the compressive strength of the concrete in which it is used in too great a degree to warrant the neglect of the consideration of this influence, while even for the same stone, the strength of the concrete will be greatly influenced by the uniformity or nonuniformity of the grading.

Again, for equally good grading, the material which grades down from the larger particles will nearly always show the greater strength. The effect of a large amount of dust in decreasing the strength of the concrete should not be overlooked. It must be recognized that the quality of the sand used in making the concrete—i.e., its hardness, size and grading, the presence of a large amount of very fine material—are all elements which will modify conclusions as to the excellence of a given stone or gravel.

From a consideration of the above facts it is always advisable to make careful investigation of the materials available, even though the costs of tests seem excessive.

The weight per cubic foot for a 1:2:4 stone and gravel concrete may be taken as 150 lb. and a cinder concrete 120 lb.

The ultimate strength of 2000 lb. per sq.in. for a 1:2:4 concrete at the age of four weeks, frequently the basis for unit stresses, was exceeded by almost 50% for the granite and gravel and by 25% for the limestone concrete, but the value found was 20% less than 2000 lb. per sq.in. for the cinder concrete.

The ultimate compressive strength of concrete shows a substantial increase from an age of four to one of 52 weeks, the strength at four weeks being about 65% of that of 52 weeks.

The yield point at the age of four weeks averages about 0.30 the ultimate strength and for stone and gravel concretes is well above 500 lb. per sq.in., but for cinders is somewhat less. The yield point shows a substantial increase with age.

The compressive stress, gross deformation diagram of concrete may be represented by a straight line up to a unit stress 1.5 times the yield point with a maximum error of about 6.5%. A working formula for reinforced-concrete in flexure, based on a constant stress deformation diagram for concrete in compression, will, therefore, give correct results up to an extreme unit fiber stress of 0.30, the ultimate strength. This would correspond to a factor of safety of about 3 and for a unit fiber stress 50% greater the error will not exceed 7%.

The initial modulus of elasticity at the age of four weeks exceeds 2,500,000, the amount usually specified, by about 85% for gravel, 70% for granite, and 45% for limestone concrete, but is 35% less for cinder concrete. The increase in stiffness with age as measured by the modulus of elasticity, while marked, is not nearly so great as the increase in strength.

The ultimate strength of concrete in compression as given by the cylinder test is always less than that given by the cube test, the ratio being about 75% at the age of four weeks, and showing a substantial increase with age, due to the greater percentage increase of strength for the cylinders.

REINFORCEMENT—A study of the 1260 tests of the steel used as reinforcement for the beams herein recorded shows that: (a) For even practically identical ultimate strengths there is considerable variation in the percentage of elongation and reduction of area. (b) The yield point bears no relation to the ultimate strength.

Transportation of Explosives in Passenger trains is contrary to Federal law, yet the Pennsylvania R.R. finds that it has been common practice in mining districts and in the vicinity of construction work for certain passengers to carry blasting powder and dynamite in their baggage. The chief offenders appear to be foreigners who are not familiar with the language or the laws. The railway company has begun a campaign to put an end to the practice, and for that purpose has caused to be posted in all stations of the company warning notices, and circulars in foreign languages have been spread broadcast throughout the mining districts. The Federal law forbids a concealed shipment of explosives or the transportation of explosives on passenger trains under any circumstances. The penalty for violations of the law is a fine not to exceed $2000, imprisonment not to exceed 18 months, or both fine and imprisonment.

and the markedly greater energy required.

The results of the punching and shearing tests of mild-steel plates tested for purposes of comparison with the rolled zinc show the following average values: Shearing stress developed, 50,040 lb. per sq.in. for the punching test, and 43,270 lb. per sq.in. for the shearing test; energy required, 25,200 in.lb. per sq.in. per inch thickness for the punching test and 16,910 in.lb. per sq.in. for the shearing test. These results do not differ widely from those of other tests of mild steel in punching and shearing.

An examination of the results of the punching tests shows that the larger the punch for any given thickness of plate, the less the unit energy required to punch the plate. In both punching and shearing tests, the maximum stress developed was slightly increased under increased speed of punching or shearing tool. Both punching and shearing tests were made with flat-faced tools, the object being to bring the stress as uniformly as possible on all parts of the area under shear. By the use of beveled punching and shearing tools, the maximum force resisting shear would have been reduced.

Ductility and Elasticity—In the series of tests described in this bulletin the elongation and the reduction of area after rupture in tension and the results of the cold-bending tests were all used to throw light on the ductility of the zinc. Unfortunately, there is no well defined quantitative standard for the comparison of the ductility of different metals. It is evident, however, from these results that zinc is much less ductile than wrought iron or mild steel and that it is less ductile across the grain than with the grain. For a zinc plate which is to be stamped or bent into shape (for example in the making of zinc elements for dry batteries) a severe cold-bending test would seem to be of considerable value in determining the acceptability of a shipment of zinc plate.

The researches of Martens and of Meyer on the effect of the heat treatment of zinc on its strength and ductility indicate the desirability of measuring and controlling the temperature in the rolling process and show the danger of rolling at too high a temperature. While the ductility of zinc is low as compared with that of steel, it is evident from the low and poorly defined elastic limit, from the loss of energy in "mechanical hysteresis" and from the behavior of compression test pieces that the zinc possesses a relatively high degree of elasticity.

Record for Culebra Cut Excavation—The highest daily record of excavation in the Culebra Cut proper, Panama Canal, was made on Apr. 11, when 44 steam shovels excavated 68,505 cu.yd. in a working day of eight hours. This is an average of 1550 cu.yd. per shovel.

United States Government Tests on Reinforced-Concrete Beams

The Structural Materials Laboratory of the United States Geological Survey was founded at St. Louis, Mo., in 1904, in connection with the world's fair being held there at that time. At this laboratory there was carried on for six years extensive tests of various structural materials, including a number upon various kinds of concrete and reinforced-concrete specimens. The scheme of the laboratory in these latter tests was to proceed from the simplest forms of cement, mortar and concrete through concrete tension and compression tests to various reinforced-concrete beams and columns. In July, 19[illegible] laboratory [illegible] Louis to P[illegible] tion of th[illegible] United Sta[illegible] under whos[illegible] be carried o[illegible] tests on cor[illegible] letin No. 3[illegible] Survey, whi[illegible] land-cement [illegible] materials, a[illegible] U. S. Geolo[illegible] the tests of [illegible] next step in [illegible] plain reinfo[illegible] completed at [illegible] the data for [illegible] the U. S. Bu[illegible] laboratory [illegible] been just pu[illegible] "technologic [illegible] 200-page larg[illegible] tude of table[illegible] impossible to [illegible] we give belo[illegible] contents.

The tests c[illegible] pression tests [illegible] 6-in. cubes ar[illegible] with varying [illegible] ment. The se[illegible] (three of which were not tested) together with the corresponding cylindrical and cubical test pieces for compression tests and 48 bond test pieces to determine the adhesive strength of the aggregates used. Representative material of four classes of aggregate, gravel, crushed granite, crushed limestone and cinders, were used in making up the test pieces which were tested in triplicate at the ages of 4, 13, 26 and 52 weeks, respectively. The report says:

> The report of these tests has been confined exclusively to a presentation of test data in such a way as to indicate with considerable certainty the various elements which affect the design of architectural and engineering structures. Studies have, therefore, been made of the maximum load between external load conditions, as measured by the bending moment, and resistance of the beam as determined by the resisting moment. There is included the relation between the unit stress at maximum load and the yield point of the reinforcement.
>
> Studies have been made for maximum load and for unit stresses of 16,000 and 32,000 lb. of the relation between the effective percentage of reinforcement and (1) the resistance of the beam as measured by $\frac{M}{bd^2}$, (2) percentage depth to the neutral axis, (3) unit compressive stress in the extreme upper fiber, (4) the deflection at the center of the beam. The results are presented diagrammatically and in greater detail in [illegible]

[illegible] the percentage of reinforcement. Finally, in this portion is a discussion of the compression tests, taking up the influence of age on relative strength of cubes and cylinders and upon the modulus of elasticity and yield point and the ratio of the yield point to the ultimate strength.

The variation in the constants entering into the composition of the beam and the completeness of the record of the tests are such that a great number of tables and diagrams are needed to present the results. At the same time, these results are most comprehensive and permit of a thorough study of the behavior of simple reinforced-concrete beams.

Conclusions

As a result of the tests, the authors have made the following conclusions:

*Technologic papers of the Bureau of Standards, F. W. Stratton, Director. No. 2, The Strength of Reinforced-Concrete Beams; Results of Tests of 333 Beams (First series), by Richard L. Humphrey and Louis H. Losse (June 27, 1911). Washington, Government Printing Office, 1912.

LIGH
PRIN

REINFORCED CONCRETE IN FLEXURE—The unit elongation of the extreme lower fiber at the time of the occurrence of the first crack is 0.00012, for stone and gravel concrete, 0.00018 for cinder concrete, and equals the unit elongation of the lower fiber at rupture of a beam of the same concrete without reinforcement.

The unit elongation of the extreme lower fiber at time of the occurrence of the first visible crack increases with the percentage of reinforcement.

The distance between cracks due to tension varies with the percentage of the reinforcement, slightly with the kind of aggregate, and is independent of the age of the concrete.

The ratio of the unit stress in the reinforcement at maximum load to the yield point varies with the percentage of the reinforcement, is independent of the

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crete as influencing a compression failure, and the relative stiffness of the concrete for varying degrees of stress as determined by the character of the stress deformation diagram of the cylinders (fixed by the proportions, consistency, kind and age of concrete).

For all practical purposes the percentage depth to the neutral axis for a given unit stress in the reinforcement may be taken as independent of the age of the concrete.

The character of the concrete as measured by its modulus of elasticity exerts an influence on the position of the neutral axis.

For any given percentage of effective reinforcement the moment of resistance varies as the ordinates of a straight line, and based on the moment of resistance of a plain concrete beam the moment of resistance for a reinforced-concrete beam is proportional to the effective percentage of reinforcement.

The moment of resistance is appreciably affected by the character of the aggregate used and the age of the concrete.

For a given unit stress in the reinforcement the deflection increases with the percentage of "effective" reinforcement and increases as the modulus of elasticity and ultimate strength of the concrete decreases.

The deflection of a reinforced-concrete beam increases in direct proportion to the increase in the unit stress in the reinforcement.

The unit compressive stress in the extreme upper fiber varies with the percentage of effective reinforcement as the ordinates to a straight line.

Excepting cinder concrete for the earlier ages and low percentages of effective reinforcement, the unit stress of 16,000 lb. per sq.in. in the reinforcement corresponds to a unit compressive stress in the upper fiber of at

which it is used in too great a degree to warrant the neglect of the consideration of this influence, while even for the same stone, the strength of the concrete will be greatly influenced by the uniformity or nonuniformity of the grading.

Again, for equally good grading, the material which grades down from the larger particles will nearly always show the greater strength. The effect of a large amount of dust in decreasing the strength of the concrete should not be overlooked. It must be recognized that the quality of the sand used in making the concrete—i.e., its hardness, size and grading, the presence of a large amount of very fine material—are all elements which will modify conclusions as to the excellence of a given stone or gravel.

From a consideration of the above facts it is always advisable to make careful investigation of the materials available, even though the costs of tests seem excessive.

The weight per cubic foot for a 1:2:4 stone and gravel concrete may be taken as 150 lb. and a cinder concrete 120 lb.

The ultimate strength of 2000 lb. per sq.in. for a 1:2:4 concrete at the age of four weeks, frequently the basis for unit stresses, was exceeded by almost 50% for the granite and gravel and by 25% for the limestone concrete, but the value found was 20% less than 2000 lb. per sq.in. for the cinder concrete.

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The Design of a Brick Chimney

By J. Norman Jensen*

This article gives the general principles to be applied in the design of an ordinary brick chimney, and presents in detail the computations necessary for the design of a typical brick chimney 125 ft. in height.

*Architectural Engineer, Department of Buildings, Chicago, Ill.

Given the required diameter and height of a chimney, there are three ways of building it:

1. Masonry construction, whose stability depends on the dead weight.

2. Reinforced-concrete construction, in which the tension in the chimney is taken care of by the steel.

3. Self supporting steel construction anchored to the foundation.

The third type of construction is temporary in its nature. The second type is still in the experimental stage. The first type is the one under consideration.

Brick chimneys are usually built of square, octagonal, or round section. The square section is rarely employed except for small heights, as it is generally uneconomical. Since the introduction of octagonal brick for the corners, the section of octagonal shape is easily built. The round section is the most common. It is usually built of specially formed brick generally known as "radial brick." Common brick is also used, but when the diameter becomes less than 5 ft. it is not advisable to use such brick. This is because of the fact that bricklayers will not clip the corners of common brick so as to obtain a good joint, but will lay a full brick instead. As a result the vertical joints on the outside face of the chimney become too wide.

If the chimney is at a corner of a building the lower part is usually made of square or octagonal section so as to better fit into the plan of the building.

As a first step in the design it is necessary to know what unit wind load to use. This is established in cities by building ordinances. According to the ordinance of the City of Chicago, "all chimneys shall be so designed as to safely resist a wind pressure of 30 lb. per sq.ft. of surface exposed to the action of the wind." This value includes whatever suction there may be on the leeward side of the chimney. Some engineers use a value of 50 lb.

In this connection it may be interesting to know that the rule (generally known as Smeaton's Rule) used at the Smithsonian Institution, at Washington, for determining the pressure of the wind when the velocity is known is as follows:

> Divide the square of the velocity in miles per hour by 200; the quotient is the pressure in pounds per square foot.

The records of the Weather Bureau of Chicago show that some time during each year the wind attains a velocity of 72 miles an hour. At intervals of about 15 years the velocity may attain 84 miles an hour. This extremely high velocity continued for 10 hours, at a certain date in the records. It would seem that a value of 30 lb. is reasonable for Chicago according to Smeaton's Rule, and that a value of 50 lb., equivalent to the velocity of a violent hurricane at 100 miles an hour, is somewhat high even for Chicago, the Windy City.*

Experiments on the actual pressure on a chimney show that the pressure may be higher than 30 lb. An empirical formula expressing this increased wind pressure for chimneys over 150 ft. in height is

$$w = 24.3 + 0.038\,H,$$

in which

w = Wind pressure per sq.ft. on a vertical plane,

H = Height in ft.

The intensity of the wind at all levels is assumed to be uniform, and the wind is considered as blowing horizontally. In some cases adjacent buildings may protect a chimney somewhat, but such protection is usually neglected.

A plane surface perpendicular to the direction of the wind is taken as the exposed surface. It is generally called the projected area. For polygonal chimneys this surface is taken at right angles to two opposite sides.

In the early part of this article it was stated that square chimneys are rarely built to great heights. This is accounted for by the fact that if the projected area of a square, octagonal, or round chimney is represented by the value F, then the total wind pressure on the chimney is taken at:

For square chimneys..... 1.00 wF
For octagonal chimneys .... 0.71 wF
For round chimneys ..... 0.67 wF.

It is evident that considering wind only there is a decided gain in economy by using a round chimney.

In order to show how these values for total wind pressure are obtained it will be necessary to study the effect of wind on any inclined surface.

In Fig. 1, let

MN = In plan any inclined surface of unit height and length;

$M'N'$ = Corresponding surface on the other side of the chimney;

θ = Angle between surface and a line normal to the direction of the wind;

ON = Projected area of the inclined surface. Length of ON = 1 ft.;

d = Distance between parallel faces of chimney;

P' = Effective pressure per unit height for the surface considered;

P = Summation of the effective pressure on all the surfaces exposed to the wind.

Referring to Fig. 1, let AB represent the total force blowing on the inclined surface. AB can be resolved into components BC and AC respectively parallel and normal to MN. Since this surface is assumed to be perfectly smooth, the component BC slides along and is lost. The normal component AC may in turn be resolved into components DC and AD respectively perpendicular and parallel to the direction of the wind. Component DC is neutralized by the corresponding component on the other side of the chimney. Component AD therefore represents that part of the original force which actually tends to overthrow the chimney. This component AD will be denoted by the letter P', and is defined above.

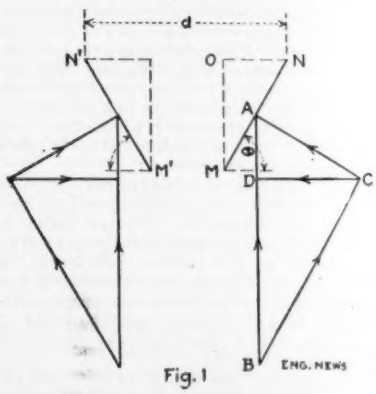


Fig. 1

From the figure,

$$AC = AB \cos\theta$$

and

$$AD = AC \cos\theta = AB \cos^2\theta$$

The force AB equals the product of the unit wind load times the projected area ON, or $AB = wl'$.

But 1 ft. $= l \cos\theta$, therefore

$$AB = wl \cos\theta$$

Since $AD = P'$, therefore

$$P' = AB \cos^2\theta,$$

or

$$P' = wl \cos^3\theta \qquad (1)$$

It is seen from the above equation that

*As explained in a recent issue of Engineering News (Mar. 7, 1912, p. 454). Weather Bureau wind-velocity records are far in excess of actual velocities, so that the 30-lb. value noted is undoubtedly conservative.—Ed. Eng. News.

when the side is parallel to the direction of the wind, that is when $\theta = 90°$, $P' = 0$. When $\theta = 0°$, that is when the side is normal to the direction of the wind, $P' = wl$.

In Fig. 2 the direction of the wind is taken normal to the side of a square chimney. Since $\theta = 0°$, $d = 1$, therefore

$$P = P' = wl \cos^2 0° = 1.00\ wd.$$

The wind is assumed blowing obliquely at an angle of 45° in Fig. 3. $d = 1$, and $\theta = 45°$. Then since the wind is blowing on both oblique faces

$$P = 2P' = 2\ wl \cos^2 \theta = 2\ wd (0.707)^2 = 0.707\ wd.$$

Referring to Fig. 4, the wind is taken as blowing perpendicular to the face of an octagonal chimney. In this case the total force of the wind is expended on one normal side, two oblique sides, and two sides parallel to the direction of the wind. The effect on the last two sides is nil. Sine $\theta = 0°$ for the normal side, and 45° for the oblique sides,

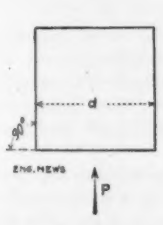


Fig.2

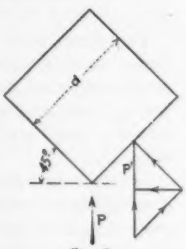


Fig.3

$$P = \Sigma P' = wl \cos^2 0° + 2\ wl \cos^2 45° = wl + 2\ wl\ (0.707)^2 = 1.707\ wl.$$

But the length of the side of an octagon in terms of the short diameter is

$$1 = d \tan 22\tfrac{1}{2}°.$$

Therefore

$$P = 1.707\ wd \tan 22\tfrac{1}{2}°,$$

or

$$P = 0.707\ wd.$$

In the analysis of a circular chimney it will be necessary to use the calculus. The general equation, $P' = wl \cos^2 \theta$, still remains true, but the differential length of an arc must be used. The length of this arc in terms of the radius is $d/2\ d\theta$. Referring to Fig. 5, it is seen that θ varies from 0 to $\frac{\pi}{2}$. The integral expression for P, therefore, is

$$P = \Sigma P' = 2\int_0^{\frac{\pi}{2}} w \cos^2 \theta \frac{d}{2} d\theta = wd \int_0^{\frac{\pi}{2}} \cos^2 \theta\ d\theta = 0.666\ wd$$

In order to bring out more clearly the general features of the design of the chimney, the detailed computations of a typical ordinary chimney are given herewith. A section and general description is given first, followed by a strain sheet showing at each offset in walls, at bottom of chimney, and at bottom of foundation: (1) Total dead weight; (2) total wind pressure; (3) wind moment; (4) section modulus; (5) area of section; (6) maximum compression; (7) maximum tension.

The notation adopted, and the formulas used in the strain sheet are given below.

NOTATION

D_1 = Outside diameter of shaft at top of section.
d_1 = Inside diameter of shaft at top of section.
D_2 = Outside diameter of shaft at middle of section.
d_2 = Inside diameter of shaft at middle of section.
D = Outside diameter of shaft at bottom of section.
d = Inside diameter of shaft at bottom of section.
t = Thickness of wall of section.
A'_1 = Area of circle of diameter D_1.
a_1 = Area of circle of diameter d_1.
A_1 = Area of annular ring at top of section.
A'_2 = Area of circle of diameter D_2.
a_2 = Area of circle of diameter d_2.
A_2 = Area of annular ring at middle of section.
A' = Area of circle of diameter D.
a = Area of circle of diameter d.
A = Area of annular ring at bottom of section.
V = Volume of section.
h = Height of section.
W_1 = Weight of section.
W_2 = Weight of shaft above section.
W = Weight at bottom of section.
w = Dead load per square foot.
A_w = Area of surface exposed to wind.
P = Total wind pressure above section.
x = Lever arm = distance to center of gravity of Aw.
M = Wind moment at section in foot-pounds.
S = Section modulus of section.
p = Wind load per square foot.
f_c = Maximum compression in pounds per square inch.
f_t = Maximum tension in pounds per square inch.
L = Length of side of square foundation.

FORMULAS

$$V = \frac{A_1 + A + 4A_2}{6} h \qquad (1)$$

$$w = \frac{W}{A} \qquad (2)$$

$$M = Px \qquad (3)$$

Round chimney,

$$S = \frac{A}{8}\left(D + \frac{d^2}{D}\right) \qquad (4)$$

$$p = \frac{M}{S} \qquad (5)$$

$$f_c = \frac{w + p}{144} \qquad (6)$$

$$f_t = \frac{w - p}{144} \qquad (7)$$

Square foundation,

$$S = 0.118\ L^3 \qquad (8)$$

The first formula is the prismoidal formula. It gives results a little closer than if an average section was taken. Formula (4) is a transposition of the ordinary formula for the section modulus of an annular ring. This formula is usually given as

$$S = 0.098 \frac{(D^4 - d^4)}{D}$$

The last equation may be written

$$S = \frac{0.125}{D}(D^2 + d^2)(D^2 - d^2)\ 0.785$$

But $(D^2 - d^2)\ 0.785$ is the area of the annular ring. The equation may therefore be written

$$S = \frac{A}{8}\left(\frac{D^2 + d^2}{D}\right)$$

or in the form

$$S = \frac{A}{8}\left(D + \frac{d^2}{D}\right)$$

This last equation has the advantage in being simpler, and more easily applied, as there is no necessity for obtaining the fourth power of a number. The other equations are the common formulas of mechanics.

Referring, then, to Fig. 6, the computations for a typical brick chimney are given. The general constants are as follows:

Inside diameter of chimney at top	4 ft.
Largest outside diameter	10 ft. 8½ in.
Height of chimney	125 ft.
Batter of chimney	½ in. to 1 ft.
Depth of foundation	5 ft.
Size of foundation	18 ft. 6 in.
Height of lining	40 ft.
Wind pressure	30 lb. per sq.ft.

Below are given the detailed computations for a section 25 ft. below the top

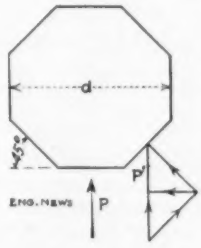


Fig.4

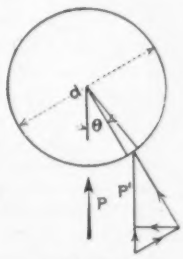


Fig.5

of the chimney and one at the foundations. In the accompanying table these figures are summarized for other sections of the chimney, the computations for each section being similar to those given for the section 25 ft. below the top.

SECTION 25 FT. BELOW TOP

$D_1 = 5.50$ $A'_1 = 23.75$
$d_1 = 4.00$ $a_1 = 12.56$
$t = 9$ in. $A_1 = 11.19$ sq.ft.
$D_2 = 6.02$ $A'_2 = 28.46$
$d_2 = 4.52$ $a_2 = 16.04$
$t = 9$ in. $A_2 = 12.42$ sq.ft.
$D = 6.54$ $A' = 33.59$
$d = 5.04$ $a = 19.95$
$t = 9$ in. $A = 13.64$ sq.ft.

$$V = \frac{11.19 + 13.64 + 4 \times 12.42}{6} \times 25 = 311 \text{ cu.ft.}$$

$$W = 110 \times 311 = 34{,}200 \text{ lb.}$$

$$w = \frac{34{,}200}{13.64} = 2500 \text{ lb.}$$

$$A_w = \frac{5.50 + 6.54}{2} \times 25 = 151 \text{ sq.ft.}$$

$$P = 151 \times 20 = 3020 \text{ lb.}$$

$$x = \frac{6.54 + 2 \times 5.50}{6.54 + 5.50} \times \frac{25}{3} = 12.1 \text{ ft.}$$

$$M = 3020 \times 12.1 = 36{,}600 \text{ ft.-lb.}$$

$$S = \frac{13.64}{8}\left(6.54 + \frac{5.04^2}{6.54}\right) = 17.8$$

$$p = \frac{36,600}{17.8} = 2060 \text{ lb.}$$

$$f_c = \frac{2500 + 2060}{144} = 32 \text{ lb. per sq.in.}$$

$$f_t = \frac{2500 - 2060}{144} = 0 \text{ lb. per sq.in.}$$

FOUNDATION

Weight of foundation = $\left(\overline{12.5}^3 + \overline{14}^2 + \overline{15.5}^2 + \overline{17}^2 + \overline{18.5}^2\right) 145 = 177,000$ lb.

Weight of shaft = 446,000

Total dead load = 623,000 lb.

Dead load on ground = $\frac{623,000}{18.5 \times 18.5} =$ 1820 lb.

$P = 19,820$ lb.

$x = 55 + 5 = 60$ ft.

$M = 19,820 \times 60 = 1,189,000$ ft.-lb.

S on diagonal = $0.118 \times \overline{18.5}^3 = 746$ sq.ft.

$$P = \frac{1,189,000}{746} = 1600 \text{ lb.}$$

Total max. compression at toe = 1820 + 1600 = 3420 lb. per sq.ft.

Min. compression = 1820 − 1600 = 220 lb. per sq.ft.

It is assumed that the height and minimum diameter of the chimney have been determined by the requirements for draft, or other considerations. The height of the firebrick lining has been determined by a clause in the Chicago Building Ordinance which states that

All flues having a greater area than 400 sq.in. shall be lined on the inside with an insulating material, which lining shall start at least 2 ft. below the smoke inlet and shall extend upward for at least ten times the diameter of the flue, or if said flue is not circular or square in cross-section for ten times the average diameter, when the flues are of brick, stone or concrete, said insulating lining shall be fire-clay brick or fire-clay blocks, and if such bricks or blocks are 4 in. or more in thickness, they may be considered as a portion of the thickness required for the surrounding walls. The walls surrounding chimneys having an area greater than 400 sq.in. shall have an insulating cavity not less than 3 in. wide surrounding the inner 4 in. of fire-brick or fire-clay blocks for not less than the height required above for insulating lining and said inner core shall be built independent of the surrounding brickwork and shall be free to expand or contract.

The design of the chimney in question is based on the following specification:

In the construction of the chimney only hard, common, select brick, laid up in good lime and cement mortar should be used. The brick should be of such hardness that it should not crush at less than 2300 lb. per sq.in. of gross area. The bond of all brickwork should be formed by laying one course of headers for every three courses of stretchers. Face-brick veneering should only be considered as part of the wall, if it is connected to wall by full headers every fourth course. No bat headers should be allowed.

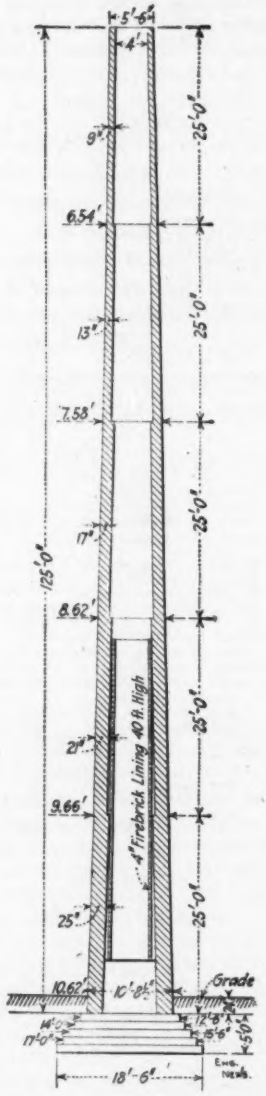


FIG. 6. DETAILS OF A 125-FT. BRICK CHIMNEY

Having determined the kind and quality of the brickwork, it is necessary to find out the thickness of wall at various points. Theoretically, the thickness of the upper part of the chimney may be very small because the stresses are usually low. Practical reasons and the result of experience have shown that the upper 25 ft. of a small chimney should not be less than 9 in. thick, and the next 25 ft. not less than 12 in thick. A convenient rule for this size chimney is that the wall should be increased 4 in. in thickness for every 25 ft. measured downward. When the inside diameter is greater than 5 ft., the upper part of the wall is usually built 13 in. thick.

The stresses in the brickwork will depend in great part on the batter given the chimney. If the batter is large the stresses will be small, and *vice versa*. A common batter is ¼ in. to the foot, and has been used in this design. This gives a ratio of width of base of chimney to height of 1 to 11.6. For ordinary round chimneys this is a good proportion. For square chimneys this ratio usually runs about 1 to 10. The above rules of thumb are only to be taken as a rough guide as to what the correct proportions should be, and are convenient in a preliminary design.

The dimensions of the round chimney under consideration are given above. It is the intention to analyze the stresses for the assumed section, and determine whether they are within safe limits or not.

In the computations of stresses the chimney is considered as a hollow cantilever beam. This beam is acted upon by the forces of wind and gravity, the single or combined effect of which tends to: (1) Overthrow a part of the chimney; (2) crush the masonry at the leeward side of any section of the chimney; (3) produce too great pressure on the soil under the leeward side of the foundation.

The first cause of failure is prevented by having the walls sufficiently thick, especially in the upper part. The second is prevented by increasing the section whenever the compression becomes too great. The third is prevented by spreading the foundation so that there could be no possibility of the chimney getting out of plumb due to the inclination of the footing.

For the masonry specified above the allowable compression has been taken at 150 lb. per sq.in. The allowable tension has been taken at one-eighth of the compression, or 19 lb. per sq.in. These values for compression and tension have been used on the assumption that the neutral axis is in the center of the chimney. This assumption does not represent the true state of affairs. Instead of passing through the middle of the shaft, the resultant of the weight of the chimney is eccentric whenever the wind blows. The amount of this eccentricity for any section depends on the bending moment for the section.

It is not good engineering to consider tension in masonry, so it must be as-

TABLE SHOWING STRESSES IN CHIMNEY IN FIG. 6

	Distance in Ft. of Bottom of Section from Chimney Top				
	25	50	75	100	125
W (lb.)	34,200	84,800	166,800	278,300	446,000
w (lb. per sq.ft.)	2,500	3,840	5,200	6,400	7,900
Aw (sq.ft.)	151	327	530	758	991
P (lb.)	3,020	6,540	10,600	15,160	19,820
x (ft.)	12.1	23.6	34.6	45.5	55.0
M (ft.lb.)	36,000	154,000	366,000	689,000	1,090,000
S	17.8	31.6	50.0	74.0	103.5
p (lb. per sq.ft.)	2,060	4,870	7,320	9,300	10,500
f_c (lb. per sq.in.)	32	60	87	109	128
f_t (lb. per sq.in.)	0	7	15	20	19

sumed that the section opens up on the windward side to a line somewhere between the windward side and the center of the shaft. The location of this line can be calculated when the eccentricity is known. Instead of the compression being distributed over the whole section, therefore, it is only over part of it. The area of this reduced section can be calculated by this exact method. As a result the compression as figured by the exact method is considerably larger than the assumption on which this design is based, and there is really no tension on the section when the exact method is employed. But the computations of the exact method are so laborious that it ordinarily is not used, and the easier assumption of the neutral axis in the middle of the shaft is used. In this method values of unit compression and tension are employed which would give a safe design if the exact method were used. In no case is the allowable tension for any masonry to exceed 25 lb. per sq.in.

It is believed that the computations submitted show the method used in determining the stresses at each section. Knowing the outside and inside diameter of the top, middle, and bottom of each section, the volume of the section is computed by the prismoidal formula. In determining the weight of the section, a unit weight of 110 lb. per cu.ft. has been used, as this represents the weight of common brickwork in and about Chicago. By adding all the weight above the section, the total weight at the bottom of the section is obtained. Formula (2) gives the dead load per square foot. In a chimney the projected area is a trapezoid, and the total wind pressure is computed by multiplying this area by 20. As explained above, the unit wind load for a round chimney is ⅔ of 30 lb., or 20 lb. per sq.ft. By using the ordinary formula for the center of gravity of a trapezoid, the lever arm of the total wind pressure is computed. The application of formulas (3), (4) and (5) gives the wind moment, the section modulus, and the wind load, respectively. The maximum compression on the section equals the sum of the dead and wind loads, and the maximum tension equals the difference between the two.

The foundation was designed so that the soil pressure due to the weight of the chimney and the foundation, and the pressure of the wind did not exceed 3500 lb. per sq.ft. In computing the weight of the foundation, the weight of plain concrete was taken at 145 lb. per cu.ft. The dead load on the ground was obtained by dividing the total dead load by the assumed area of the foundation. The total wind pressure was the same as the section at the base of the chimney, but the lever arm was figured from the bottom of the foundation. The depth of the foundation was so made that it was well below the frost line, and so that the offsets in the foundation should make an angle of 60° to the horizontal.

The section modulus of the square foundation was taken on the long axis instead of on an axis parallel to a side, as the first condition gives a greater compression on the soil. The wind load on the soil was obtained by dividing the wind moment by the section modulus. The total compression at the toe of the foundation equals the sum of the dead and wind loads. The difference between the dead and wind load was positive, showing that there was compression over the entire area of the foundation. If this last result had been negative, it would have shown that there was an unlifting tendency, which good design will not allow.

In conclusion it may be noted that the limits set for tension really control the design of the chimney shown, as the values for compression are always within safe limits. The soil pressure does not exceed the limits set, and other requirements have been satisfied as to the design of an ordinary brick chimney.

A Suit for Royalties alleged to be due for the use of a gas check on breech-loading cannon has recently been decided by the U. S. Supreme Court in the case of the United States vs. the Société Anonyne Des Anciens Etablissements Cail. The judgment against the United States for $136,000 made by the Court of Claims was confirmed by the Supreme Court. It appears that Col. De Bange, an officer in the French army, in 1883 patented the use of a pad of asbestos and tallow back of a stout-headed sliding pin in the screw plug of the breech mechanism, and this was brought to the attention of the U. S. War and Navy Department officials. Similar devices have been used in the U. S. Army and Navy since then and have been generally known in the service as De Bange gas checks. They are described as follows in "Modern Ordnance in the U. S. Navy," 1894 (from the record of the case):

> The De Bange gas check, used in this and all other guns without metallic cartridge cases, consists of a steel mushroom with head nearly of the diameter of chamber, and a shank passing through a central hole in the breech plug and an obdurating pad on the shank between the mushroom head and the face of the plug. The obdurating pad consists of two steel disks inclosing a canvas-covered pad of asbestos and mutton tallow. Two nuts on its inner end hold the mushroom in place and a frictionless washer between the forward nut and the plug allows the latter to turn independently of the mushroom when the pad sticks to the chamber walls after firing. On firing the mushroom head is pressed back and forces the asbestos ring tightly against the surface of the tube completely sealing all escape of gas.

The attention of the War, Navy and State departments was drawn to Col. De Bange's claims for compensation in June, 1891. The final outcome of the departmental investigations was that questions arising under the patent and growing out of the claims and rights of other patentees were involved, and that the determination of all these questions was to be ascertained by the Court of Claims rather than by the department. The government appealed from the findings of the Court of Claims, contending that there was no implied contract between the claimant and the government since the mere use of the invention with the knowledge of the patentee did not create an implied contract to pay for such use. It was claimed that the arrangement adopted differed in important respects from the one shown in De Bange's patent, so that there was no infringement. The record also showed that there had been no authority of law until about 1894 for the payment of royalties out of the departmental appropriations.

The Supreme Court finds that the officers of the government knew of the great importance of the invention, and that their purpose deliberately to take the property of another without compensation cannot be imputed to them without the most convincing proof. The Court found that these officers, moreover, contemplated compensation but not the amount and extent of it. The Supreme Court supported the findings of the Court of Claims and held that the use of a tallow and asbestos pad was the central idea of the De Bange invention and that infringement depended on the use of this idea and not on the particular arrangement.

Safety Guards for Punch Presses are discussed in the "Journal of Industrial Safety" for March, 1912. Five different safety devices are described for protecting the operator of the press from injuries to the fingers or hands. The first type has a metal finger or arm which sweeps over the die as the ram descends and is depended upon to push the operator's hand from the danger zone. The second type has a telescoping cylindrical guard which lies close upon the blank being fed to keep the operator's fingers outside the danger zone. The third type, which is sometimes designated as the interlocking system, aims to secure safety by requiring that both hands of the operator be used to engage the mechanism causing the stroke of the press. Two hand levers are provided both of which must be moved to engage the clutch.

The fourth system consists of a light metal frame attached to the levers by which the clutch is engaged. This framework descends upon the work table when the pedal operating the clutch is depressed. If the fingers or hands of the operator are underneath this frame, so that it is unable to touch the work table, it is not possible to depress the pedal far enough to engage the clutch. This device can be adjusted so sensitively that the presence of even an object no thicker than an ordinary lead pencil will prevent the engagement of the clutch. The frame must be down close upon the die before the clutch can be thrown in and the ram descend. This has the advantage of not interfering with rapid production. If the press is being operated continuously, the device can be arranged so that the clutch will be thrown out if the ram starts to descend when the operator's fingers are beneath the safety frame.

The fifth device involves an electrical connection with the die plate so that if the operator's fingers are resting on the plate when the ram starts down they will receive a sharp shock. The involuntary contraction of the muscles is depended upon to pull the endangered fingers out of the path of the ram. The electrical circuit in this device is completed by contact points brought together by the motion of the mechanism which throws the clutch into action.

Arthur's Pass Tunnel, New Zealand*

By W. H. Gavin†

This 5¾-mile railway tunnel through a mountain range is being driven by American machinery and partly under the direction of American engineers, although the English bottom-heading system of tunneling is employed. A special feature is the use of concrete blocks in the arch lining. The execution of the work is described in some detail by the engineer who is in charge for the contracting company.

This tunnel is now being constructed for the New Zealand Government to give eastern railway connection to coal lands in the province of Westland. It pierces the mountain range known as the Southern Alps at a height of 2400 ft. above sea level. The tunnel is 5 miles 1660 ft. long, all on tangent, with a 3% grade from the west end to the summit at the east portal as shown in Fig. 2. It carries a single-track line of 3½-ft. gage, which is the standard gage of the New Zealand Government railways.

In cross-section, the clear height above subgrade is 16 ft. 9 in.; maximum width, 15 ft. The standard cross-section is shown in Fig. 3, and the alternative section shown by the heavy dotted lines at the left is for heavy ground; this latter section has not yet been used. For the normal section with 12-in. lining, the excavation is 10 cu.yd. per foot run of tunnel; actual yardage, say 11½ yd. The tunnel is being lined throughout with concrete sidewalls and concrete block arch. Refuges or recesses for maintenance-of-way men are provided on the left-hand side at intervals of 330 ft. Their normal size is 7 ft. high, 6 ft. wide and 2½ ft. deep; but at ¼-mile and ¾-mile intervals they are 7x8x4 ft. and 8x10x6 ft., respectively.

The tunnel is in solid rock, except for a few hundred feet at the portals. The country rock lies on its edge in more or less vertical beds of greatly varying thickness, whose strike is more or less parallel to the tunnel. The maximum variation in dip is about 45° right or left of the vertical, and in strike 45° right or left of the center line. The rock is jointed in all directions, and is fissured badly. It shoots fairly; but is gritty and hard on the drill steels or rods. It changes abruptly from very hard close sandstone, medium sandstone and indurated slaty shale, giving from 4 to 24 in. of boring per steel. It commonly stands without timbering until permanently lined, but may require repeated scaling. Temporary timbering is used in places to prevent flaking.

FIG. 1. THE SUMMIT OF THE ARTHUR'S PASS WAGON ROAD IN NEW ZEALAND

(Elevation, 3050 ft., or 615 ft. above the summit of the new railway tunnel.)

*The Arthur's Pass Tunnel (or Otira Tunnel) is on a new line of the New Zealand Government Ry. (in the South Island), which will extend from Rolleston to Stillwater, and connect Christchurch on the east coast with Greymouth on the west coast. It will thus give direct rail communication between the eastern and western sections of the island, which are separated by the Southern Alps. A highway crosses the range and has its summit in the Arthur Pass (over the tunnel), where the road passes the base of Mt. Rolleston, whose crest is about 9000 ft. above sea level. The summit elevation of the pass (shown in Fig. 1) is about 3050 ft., while that of the railway (at the east portal is 2435 ft. Work on the tunnel was commenced in May, 1908. The contract period is five years, and the contract price about $3,000,000.—Editor.

†Engineer to the Contractors, Otira, New Zealand.

The ground is sometimes dry, commonly moist, and occasionally as wet as tropical rain. No stream has been struck of such size as to require special methods. The climate is temperate, but very wet. The usual rainfall is 17 ft. a year, with a monthly maximum of 36 in. The present flow of tunnel water at the west portal is 81,000 U. S. gal. per hour, being the drainage from 1.64 miles of tunnel. The greater part of the construction is being done uphill from the lower (west) end, on account of the 3% grade favoring muck traffic, and because of the heavy pumping anticipated to drain any length of tunnel at the east end. The description of the work therefore applies particularly to the west end.

BOTTOM HEADING

The excavation is by the bottom-heading method, followed by enlargement to full section. There are three 8-hour shifts six days a week; on each shift the men work 7½ hours and have ½ hour for lunch. Fig. 4 shows the general arrangement of working for the advance heading and the enlargement. The heading is 8 ft. high, 10 ft. wide for single track and 12 ft. for sidings. The sidings are 260 ft. long over the switches, with 400 ft. of single track between the switches. Two to three Ingersoll-Sergeant 3¼-in. drills are used on a 4½-in. cross-bar, without arms. Two machines have been as many as could be shoveled for in very soft ground, on account of roof overbreaking. When driving 12 ft. wide, the full width is preferably carried along with the face, by using a longer cross-bar, as widening out the heading behind interrupts traffic. There are 8 to 10 men on a shift; in hard ground (8x10 ft.), 3 machine men, 2 helpers, 2 muckers, 1 winchman; in easy ground (8x12 ft.), 2 machine men, 2 helpers, 5 muckers, 1 winchman.

The round is fired near the end of each shift. The last man out, after lighting the fuses, turns on the face air, and 20 minutes later the face is clear of smoke for 50 ft. The next shift comes straight in; the machine men and helpers trim the roof, and dig down into the muck-pile for the rig. If this pile is too near the roof, as is common in soft ground, the muckers clear the track up to the main muck pile, and bring up the machines on a light flat car. The machine men and helpers rig the bar and machines, which usually takes 20 min., but longer in unfavorable ground. The muckers bring up the first muck car, and start shoveling. Drilling starts 30 to 60 min. after commencement of shift, or 50 to 80 min. after firing. The muckers usually get all muck away in good time for the machines to bore the lifter holes; but in soft ground, the muck at the face is sometimes dug down to within 18 in. of the floor to allow the lifters to collar, and then the round is fired without completely mucking out.

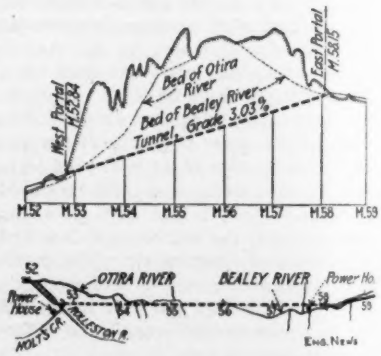


FIG. 2. PROFILE OF THE ARTHUR'S PASS TUNNEL

While boring is being finished, the muckers lay rails, bring up explosives, and take back machines, hose, steel, etc. The loading of holes begins as soon as one machine is bored out; and the round is fired (if possible) 20 minutes before end of shift, so that the next shift can

start without waiting for the smoke to clear. From 4 to 6 lb. per cu.yd. of Nobel's gelignite is used in the face. In very hard ground, 30 to 50% of this is blasting gelatine and the balance is gelignite. The usual cut-holes and safety fuse are employed. Holes are 10 to 16 in number; 4½ ft. in hard ground to 6½ ft. in easier, and with an inclination or pull of 3 to 5½ ft.

The best run has been 13½ ft. per day for 12 working days, Sundays not worked. The machines drill, without arms, off a horizontal 4½-in. bar, with double screw-jack at one end, rigged about 3 ft. below the roof. The bar is reinforced by "stuffing" it with a core of Australian hardwood. One rig or setting is commonly sufficient; but in ground that shoots badly it occasionally becomes necessary to make a second rig about 2 ft. above the floor, in order to get the lifter holes in flat, and to get the bottom into shape.

Dump Car Service

Empty cars are worked (one or two at a time) from the bottom-heading siding to the face, by an air-winch (or hoist) and wire rope, with a sheave-block attached to a vertical drill column, rigged at the side, near the face (Fig. 4). The full car returns down grade by gravity, trailing the rope for the next empty. The round trip takes 2 to 5 minutes, according to distance. The winch is placed in a chamber cut in the side of the bottom heading, just above the siding. The distance of face from winch increases from 100 ft. to 800 ft. The winch has a second drum, with a heavier rope, used for handling cars in trains lower down the tunnel.

minutes; but men and conditions vary. Not more than three men can shovel abreast in 10 or 12 ft., and shovelers in excess of that number work at a disadvantage.

Enlargement to Full Size

At such a distance behind the bottom-heading siding as will avoid interference

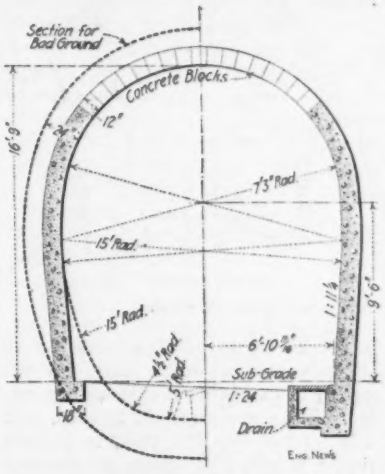


Fig. 3. Cross-section of the Arthur's Pass Tunnel; New Zealand Government Railways (3 Ft. 6 In. Gage)

(The heavy dotted lines at the left show the alternative section for bad ground, the drain here being at the center and within the lining.)

of cars (say, 300 ft. or more) a staging is erected in the bottom heading to form a floor to carry the muck produced in driving and enlarging the top-heading (Fig. 4). This staging consists of sets of round timber set across the heading at 4-ft. centers, having 10-in. round timber caps wedged against the sides of the heading supported by 9-in. round timber legs (or in narrow places 6-ft. lengths of rail split to form a Y at top) and decked with 3-in. split timber. This decking runs longitudinally and over the center of the track a slide (operated from below) is provided in each bay for loading the muck into cars.

Care is taken that there is a clear space of at least 12 in. left between decking and roof to allow for the swelling of the rock when the top-heading face is fired; otherwise a "break-through" will result. In addition, when firing the top-heading face extra legs or "toms" are put in to reinforce the caps directly under the blast; these are removed immediately after firing in order to clear the track (Fig. 5).

Top Heading

The top heading is driven 8 to 11 ft. wide by the full height from roof of bottom heading to roof of tunnel, about 11 ft. It is drilled by three Ingersoll-Sergeant 3¼-in. machines rigged without arms on a stuffed 4½-in. crossbar. The bar is rigged high, and the men commence work while standing on the muck-pile of the previous blast; they change to a plank staging as the muck-pile is lowered. From 9 to 12 holes are drilled in this face, 6 to 4 ft. deep, and the ground broken to the stage; the three lowest holes are fired first, in a separate group, to ease the staging. The muck is dropped through the chutes provided, into cars standing on the single track in the bottom heading. Three machine men, two helpers, and one mucker (who also handles steel) work this face, and do 7 to 13 ft. a day. The arrangement is shown in Fig. 4.

Not less than 60 ft. behind the top-heading face are one or two 3¼-in. machines, each rigged on the arm of a separate cross-bar, boring holes square into

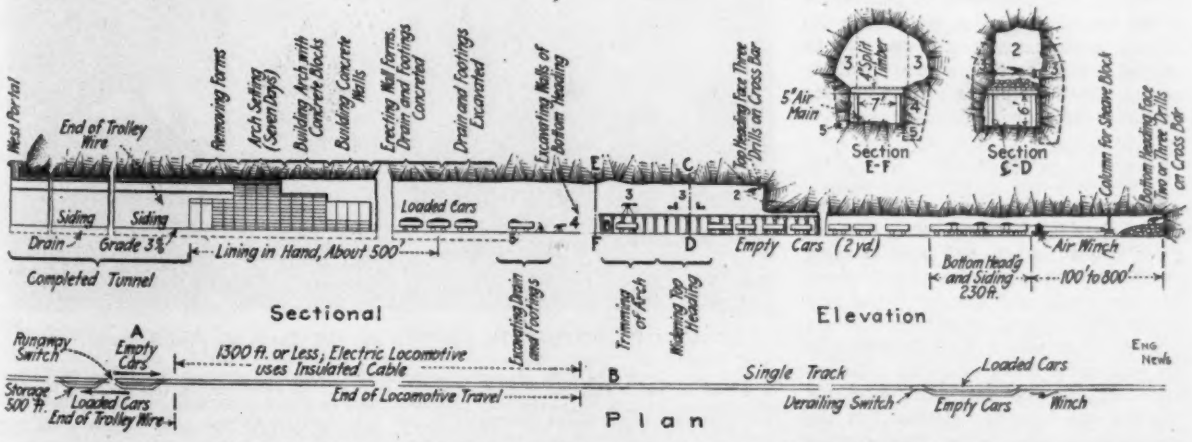


Fig. 4. Construction Methods and Plant for Driving the Arthur's Pass Tunnel

The cars are wooden, side-dump, 2 yd. flush; they are 4 ft. 2 in. high above the rail, 4 ft. 3 in. wide over all, and each is fitted with a strong screw brake acting on all four wheels. In the bottom heading it takes from 15 to 25 minutes to fill a car well heaped, or say 50 to 60 man-

the walls of the heading, to break out the arch to size. There are two men to each machine. Behind these again, say 50 ft., works one man with an air-feed stoping hammer drill (or for the flatter holes a 2½-in. piston drill on tripod), trimming the arch to a finish; there is al-

so a shoveler trimming muck to chutes, scaling roof, etc. This completes the arch. Not less than 150 ft. behind the top-heading face, there is a gang of 4 or 5 men tearing down the bottom heading staging, and sending it through ahead, as empty cars offer, to be reërected. These men also work 2½-in. drills on tripods (and part of the time an air-feed stoper) for widening out the bottom heading to full width, and they shovel the muck into cars.

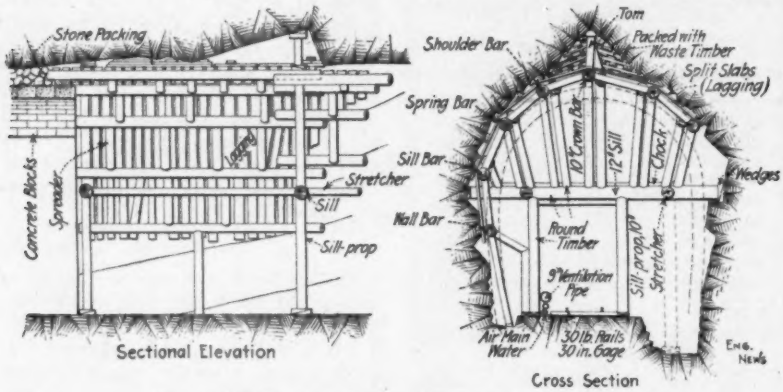


FIG. 5. TIMBERING FOR THE ARTHUR'S PASS TUNNEL

Behind these again (some 50 ft.) one man works a hand hammer drill, or 2½-in. piston drill on tripod, excavating for the 18x12-in. footing of the left-hand concrete wall. Behind him the water, which has so far run down the track all the way from the bottom-heading face, is directed into this footing excavation. Three or more men with one or two 2½-in. piston drills on tripods, and a hand hammer drill, are employed in excavating and trimming up the 3x3½-ft. drain on the right. This completes the excavation. If the bottom-heading is far enough advanced, enlargement and lining can be carried on at several points simultaneously. We have driven the top-heading at three faces and enlarged.

TIMBERING

The occasional main tunnel timbering is on the English crown-bar system, of green round timber and split lagging (Fig. 5). This system leaves good clearance for traffic and fair for lining, and allows the removal of all the timbers (except sometimes a little lagging), as the permanent lining proceeds. A complete set may be developed from the smallest top heading; placing first the crown-bar on false props; then working out the ground on each side, and poling around and placing the other bars successively, finally placing the main sill and repropping off it, and placing sill-props in the sides of the bottom heading. The great thing is to wedge the bars hard against the ground.

The sills are spaced 16½-ft. centers, or less in heavy ground, in which latter Australian hardwood (ironbark) bars and sills are preferable. All timbers, except sills, are removed without cutting, and can be used again. The sill is built into the concrete (without boxing), and must be cut to remove it. The holes left by its ends are concreted later. A sill scarfed in the middle would obviate cutting. But a one-piece sill is used, with ends hitched into the rock, to prevent trouble should a sill-prop or stretcher bar be shot out. This care is necessary, as the walls of the bottom heading, drain and footing remain to be excavated after the timbering of the arch, and the fall of a section or two of timber and rock would disorganize the whole tunnel work.

ELECTRIC HAULAGE SYSTEM

Every four hours a 10-ton electric locomotive pushes a maximum train of 28 empty cars up from the portal. It passes a loaded train at the upper siding in finished tunnel (*A*, Fig. 4), and continuing on past the lining in progress, leaves the cars under the bottom-heading staging (at *B*) where arch excavation is in hand. Thence the train is hauled forward by the air-winch until the rear car is beyond the face of the top-heading, when the rear 18 cars are detached, and the balance go on to the bottom-heading siding for use in that face. The 18 empty cars are dropped by gravity (separately or in groups) down the single track through the several points of enlargement, as required. At the end of the four-hour loading period, the loaded cars of the bottom heading are dropped down to the loaded cars at the enlargement, and all are coupled up and run by gravity to the siding *A*, in the finished tunnel. There they pass the incoming train of empties, thus completing the four-hour cycle. This is single-track working, and is all right as long as the foreman in charge looks ahead of him. Finally the cars are dumped from a trestle onto a 30-ft. fill.

At the west end, the bottom heading face is now 2½ miles from the dumping point. The main line head of track is 1¼ miles from the west portal, and is connected with it by the contractors' trolley track. Material for the east end of the tunnel is taken over Arthur's Pass by wagon from this head of track.

FIG. 6. CONTRACTOR'S LOCOMOTIVES FOR THE ARTHUR'S PASS TUNNEL

(At the right are two electric locomotives, each carrying an insulated wire on a drum, so as to run beyond the trolley line. Next to them, in the middle, is an oil engine used before the electric locomotives were installed. At the left is a small saddle-tank steam locomotive. All are of 30-in. gage.)

The motive power for the 30-in. construction line consists of two 20,000-lb., 100-hp., Jeffrey electric mining locomotives (from Columbus, Ohio). Each is fitted with a drum carrying an insulated cable, and a rewinding motor and brake. The cable is hooked to the end of the trolley wire, and enables the locomotive to run 1300 ft. beyond it (Fig. 4). This

avoids the difficulty of providing a suspended naked trolley wire in the uncompleted part of the tunnel. These locomotives had sockets for two trolley poles (one on each side) at the controller end. This was inconvenient, and a small traversing carriage on a bar frame has been rigged across in front of the motorman, to carry a socket for one pole. Thus the pole can now be pushed from side to side with one hand (and without cutting off current); this is convenient when taking a crossover, or turning the pole in the narrow tunnel.

Before the electric locomotives were supplied we used a 16,000-lb. single-cylinder, two-cycle, benzine locomotive. This was not very satisfactory, owing to uncertain action and to fumes in the tunnel. It is now kept as a reserve. No horses are used at the west end. Fig. 6 shows the two electric locomotives at the

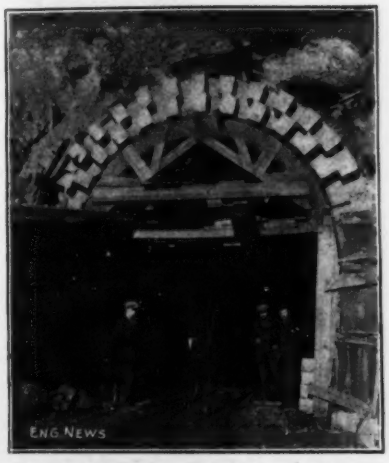


FIG. 7. ROOF ARCH WITH TWO 12-IN. RINGS OF CONCRETE BLOCKS; ARTHUR'S PASS TUNNEL

right, the oil locomotive beyond them (on the same track) and a small saddle-tank steam locomotive (0:4:2) on the left.

LINING

All lining is in concrete, mixed 1:2:5. The normal thickness is 12 in.; 18 or 24 in. being used in heavier ground. The mass concrete of the sidewalls is carried up to about 14 ft. above subgrade, and the arch is then built of solid concrete voussoirs molded outside the tunnel.* The contract provided for 25 courses of blocks in the arch. This was reduced to 17, and later to a minimum of seven courses in the crown. Fig. 7 shows the use of a double ring (24 in.) of blocks near the west portal. The concrete for walls (using 2 in. aggregate) is mixed outside in a Ransome mixer, and delivered into low cars with 11x4-ft. deck, 15-in. sides, and ends removable for shoveling. From 4 to 6 cu.yd. at a time are pushed into the tunnel by an electric locomotive between the muck trips.

Concrete below subgrade is laid in lengths of 200 to 300 ft. A sandbag dam is made in the drain, and the water diverted to the left footing. The bottom of drain is then concreted to true line and level. Collapsible forms are set on this bottom, and the sides of the drain are concreted to subgrade, and the sole plate for wall uprights is set. Water is then turned back into this drain, and the left footing is concreted and the sole-plate set. The uprights of the wall forms are then erected, and walls concreted 10 ft. high for any convenient length; then the arch-ribs are set and the mass concrete is completed.

The concrete is shoveled by four men directly from the car into the forms until the wall is about 6 ft. high. Then stages (*C*, Fig. 8) are laid, and the concrete is shoveled to them from the car and thence into the form. Spaces between the concrete line and the rock are packed with stone as work proceeds, a car or two of packing stone being sandwiched among the cars of concrete for this purpose. The arch is built from stage *D* (Fig. 8), to which concrete blocks and 3:1 mortar are hoisted vertically through a well hole from cars, or the storage pile at subgrade. The overbreak space is hand-packed with stone as in the walls.

Seven men form a gang, on mass concrete, doing 50 to 60 ft. a week (6 days); eight men on arch blocking do 60 to 100 ft. a week; and four men strike and raise forms for both. Progress depends much upon the amount of packing required. These lining methods are more efficient than they might seem. Owing to narrowness of tunnel, small yardage in work, occasional temporary timbering and the provisions of the specifications, there is difficulty in making any arrangement such as an incline or traveling stage. The specification provides that forms shall not be stripped under 14 days. Fig. 8 shows the forms for the sidewalls and the centering for the arch.

The limit of haul of concrete from outside the tunnel having been nearly reached, arrangements are in hand for mixing concrete inside the tunnel, near the work, with a ½-yd. Ransome mixer, having an elevating skip for charging from cars. The side drain, 18x18 in., is concrete lined, and has a reinforced-concrete slab cover. The slabs are 2x2 ft., 2½-in. thick, having a lug on one edge, engaging a corresponding recess in top of drain wall, to prevent movement.

CONCRETE BLOCKS FOR ARCH

The concrete blocks for normal 12-in. lining are plain voussoirs 12 in. deep, 9 in. wide on soffit and 18 in. long. A block weighs 168 lb. One man lifts them to place from stage *D*, Fig. 8. The blocks are laid in 3:1 cement mortar, with the long dimension parallel to tunnel axis, the courses breaking joint. The arch is usually built in lengths of 16 ft. Extra thickness in the arch is obtained, when required, by using two or more rings of blocks. The rings are not bonded to each other.

The concrete arch blocks are molded outside the tunnel, of 1:2:5 concrete, using 1½ in. aggregate. They are made in knock-down wooden molds with steel partitions; each mold taking three blocks end to end, and having a bottom 5 ft. long. The long curing shed has four parallel longitudinal tracks of 30-in. gage, connected at one end (by a depressed transfer track) with the track on a steel vibrating table set in front of a Ransome mixer. Each track is provided with a

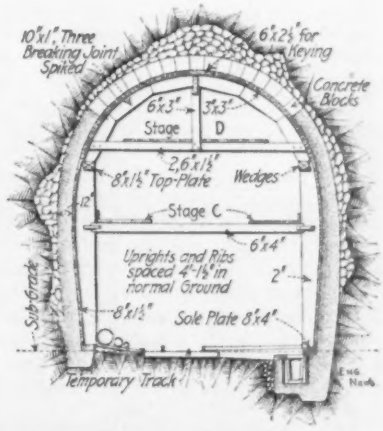


FIG. 8. FORMS FOR SIDE WALLS AND CENTERING FOR ARCH; ARTHUR'S PASS TUNNEL

pair of raised continuous skid railings, one running on each side of track, 3 ft. apart, 21 in. above rail. Four empty block molds lie crossways on a small car, the deck of which can be easily raised or lowered about 1½ in. by a lever. The car with deck raised is run onto the vibrating table, and the four molds are filled by hand. The car is then transferred to one of the parallel tracks, and run through between the skid railings to any desired point, the bottoms of molds clearing ¾ in. above the skid railings. The lever is then raised and the car-deck dropped so that the molds rest transversely on these railings, and the car (with deck ¾ in. clear below bottoms of molds) is withdrawn empty. The car is then run under four empty molds on another track, the molds are picked up by raising the deck, and the process is repeated.

In two to three days the sides, ends and partitions of molds are stripped and the blocks are left on the bottoms of the

*Concrete blocks for the arch lining of tunnels have been used on the Mexican Central Ry., and were described in Engineering News of July 26, 1906. They were used as being cheaper than cut stone. The blocks weighed 108 lb. each and were 20 in. long, tapering from about 9x8 in. at one end to 8x8 in. at the other end. The concrete was a 1:3:6 mixture, using 1-in. broken stone.—Editor.

molds on the skids for another five days. Then the car is again run under them, and by means of the rising deck (worked by one man) four bottoms with their 12 blocks are lifted clear of the skids, and run out to the storage yard. There the blocks are piled alongside tracks connected with the tunnel, to season for a further 43 days, after which they may be used.

The motor-driven vibrating table is 12x7 ft. It has a teetering vertical motion of ¾ in. at the ends, and a sideways oscillation of ⅜ in. at the same time, and makes 300 complete vibrations per minute. With a medium wet mix, this gives a good dense concrete and good faces, without any tamping. This table was built by the Improved Concrete Construction Co., of England. It is operated by a 6-hp. motor.

Concrete Materials

Stone, gravel and some sand are obtained from the wide bed of the mountain stream at the portal. These have to be washed. After rejecting boulders 8 to 36 in. diameter, the selected material is sluiced, with power tail-race water, into a 16-in. flume on 4% grade. This carries it 600 ft. to the intake of an 8-in. hydraulic elevator, Fig. 9. The elevator has 400 ft. head on a 1⅝-in. jet, and lifts material and water 35 ft. through a 9-in. riveted steel pipe into the enclosed end of a short elevated flume. An old crusher jaw is hung over the end of the pipe to deflect the stream and take the very severe wear. The flume is 24 in. wide and 8 in. deep, on a grade of 3.3%, and commands the bins, crushers and storage piles.

After rejection of larger material for crushing by means of fixed gratings, the 1½-in. material and part of the sand fall with the water into a V-shaped bin discharging at the bottom into the boot of a bucket elevator; the dirty water escapes over the top of the bin. At first it was intended to keep this bin 75% full, and draw material from it while the supply was running in at top. But owing to shortness of sand, too much water is liable to come through the discharge valve. Therefore, it is found best to draw from the bin while the supply of material and water are diverted to a storage pile kept for dry weather. It is not considered necessary to remove the whole of the sand from the aggregate.

There are two fixed gratings at steps in the high flume, and the oversize from the first grating falls to a gyratory crusher delivering to the intake of the hydraulic elevator. The oversize from the second grating falls to a jaw crusher which delivers to the boot of the bucket elevator, having a 50-ft. lift to a swinging chute which covers the several divisions of a final bin commanding mixers and cars at grade. Three to four men attend the apparatus, including crushing, and deliver the material into the final bin for an average of 65c. per cu.yd., including repairs to plant. Occasional cars of tunnel rock are dumped into the lower flume.

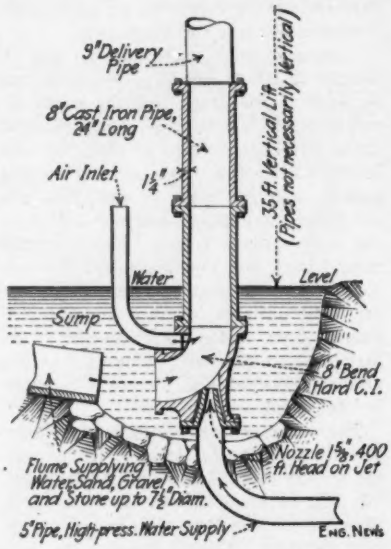

9" Delivery Pipe
8" Cast Iron Pipe, 24" Long
Air Inlet
1¼"
35 ft. Vertical Lift (Pipes not necessarily Vertical)
Water Level
Sump
8" Bend Hard C.I.
Nozzle 1⅝", 400 ft. Head on Jet
Flume Supplying Water, Sand, Gravel and Stone up to 7½" Diam.
5" Pipe, High-press. Water Supply
Eng. News

Fig. 9. Hydraulic Elevator

Sand is scarce, and a supplementary supply for concrete is carted six miles from a point on the Otira River. This is too rough for mortar, sand for which is taken from the high flume by using an "undercurrent." This is a recess 1 in. deep, 5 ft. long, and full width of flume, made in the bottom of the flume, and covered with a steel sheet perforated with ¼-in. holes. The fine sand, running along the bottom of the flume, drops with a little water through the holes into the recess, whence it is conducted by a small boxing with controllable inlet, into a settling tank. This tank has a valve at the bottom by which wet sand is drawn off without getting the overlying water.

Fig. 10. West Portal of Arthur's Pass Tunnel; New Zealand Government Railways

A good clean uniform sand is thus provided. Surplus water, carrying silt, runs away over the top of the tank.

Power Plant

At the west end is a hydro-electric plant generating 500-volt direct current. The head at the power house is 750 ft., from an 18-in. pipe ½ mile long. Two Pelton wheels of 300 hp. are coupled to two 550-volt generators, and current is carried overhead across the Rolleston River for 1500 ft. to the compressor house near the portal, where the switchboards are located. Here two 100-hp. motors drive two 2-stage Ingersoll-Rand air compressors, each with a capacity of 600 cu.ft. per min. Further compressor power is being arranged for. The 5-in. air-main gives 80 to 90 lb. pressure at the face, with all machines running and 90 to 100 lb. at the compressors. An Ajax drill sharpener is used, driven by a small independent compressor. Star bits are used.

Ventilation, Lighting and Water Supply

Ventilation is provided by a motor-driven Roöt blower of 55 hp., outside the tunnel (arranged for force or suction). This puts 3000 cu.ft. of air per minute into the tunnel, through 1¼ miles of 16-in. riveted, flanged, steel pipe extending up to the end of the finished work; beyond this a 9-in. pipe continues up to the wall excavation in progress. The arch excavation and bottom heading are blown out by air from the pressure main at the time of firing, or once a shift.

Lighting outside and in the finished part of the tunnel is by electric incandescent lamps, using two 250-volt lamps in series at each point. This voltage is too high for lighting in tunnel working places, and Star acetylene miners' handlamps are there used, one to each man. They are made in France, and are satisfactory. At both ends of the tunnel, water for drilling, drinking, etc., is supplied to all faces.

East End Workings

The plant at the east end is similar to that at the west end as to power, compressors, drills, etc., and has two 6-in. motor-driven pumps of 18,000 U. S. gal. per hour for pumping water out of the down-grade tunnel face. So far these pumps are not commonly worked to anything like their capacity. No locomotive is used at the east end, but loaded cars in trains are hauled by an air-winch out of the tunnel to the summit of the grade. Thence they run by gravity to a dumping trestle and fill, being returned by horses.

Force and Labor

The men are a fair average, but there are not nearly enough obtainable, and we have always been short. The union is in evidence, and has had the men out once or twice. The men are chiefly New Zealanders, Australians and British. They are paid as follows for eight-hour shifts: shift bosses (in charge of four to ten men), $2.90; machine men, $2.64; muckers, $2.40. In wet places, the men go out after six hours' work and are paid

for eight hours. All receive, in addition, a bonus per cu.yd. excavated, amounting to about 10c. a day. Further, a bottom-heading bonus is paid for all footage driven, over 100 ft. for 12 days of three shifts. This bonus is on a sliding scale, so that it pays per man about as follows: for 125 ft., 16c. a day; for 150 ft., 43c.; for 175 ft., 81c.; for 200 ft., $1.32.

Under the contract the men are housed in pairs in separate wooden huts, 12x12 ft., and change rooms, baths, drying rooms, etc., are provided at the portals. The average total number of men inside and outside the tunnel is 160 at the west end and 60 at the east end, but 100 more could be employed profitably to get full value out of the plant.

Progress

At the west end, the bottom-heading face is 1.64 miles from the portal, and the tunnel is lined for 1.16 miles. At the east end, the bottom-heading face is 0.35 mile from the portal, and the tunnel is lined for 0.25 mile. Progress has been slow owing to shortage of men, and occasional shortage of water power, as well as some very hard rock. Under all the circumstances, it is probable that the American top-heading system would give better economy at the west end.

Cave-in—At the east end, for 1000 ft. or more from the portal, the tunnel runs close to the river, under a steep hillside, and with very little cover in places. In May, 1910, at a point where the arch was within 30 ft. of the surface, and the roof was very thin rock covered with clayey gravel, the weight of ground broke through the timbering of completed excavation, for 50 ft. along the tunnel, and ran to the surface. Some men were caught in this fall, one of whom died later. Two men, free and unhurt, were imprisoned in the bottom heading, beyond the break, for four days, while an adit was driven from the river side to get them out. Conversation with them was had through the 5-in. air main, through which also they were provided with dry clothes and food. They were none the worse for their experience.

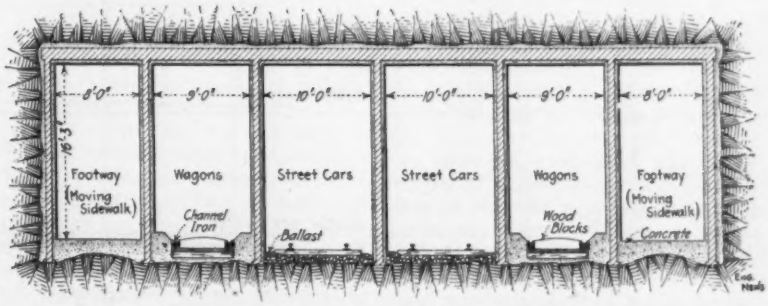
Fig. 10 shows the portal of the west end of the tunnel, which is of mass concrete.

Engineers and Contractors

The government Engineer-in-Chief is R. W. Holmes, M. Inst. C. E., Public Works Department, Wellington; and his assistant engineer in resident charge is Hugh Dickson, Public Works Department, Otira. The consulting engineer on the general tunnel scheme and location was V. G. Bogue, New York. The writer is engineer to the contractors, and was previously assistant engineer on the Flower Pass tunnel of the Western Pacific Ry. (in Nevada). The contractors are John McLean & Sons, Limited, of New Zealand. The contract is a lump-sum contract, based on schedule rates for net cubic quantities, which do not include overbreak, or packing behind lining, which accordingly are done entirely at the contractors' expense. The contractors are made responsible for alignment and grade. The time for completion is five years from signature of contract.

A Proposed Highway Tunnel in San Francisco

A highway tunnel in San Francisco, to be run under Devisadero St. from Sacramento to Greenwich, is being agitated by the Devisadero Street Improvement Association. This tunnel would connect what are locally known as the "Happy Valley" and "Harbor Hill" districts and would be 2950 ft. long (with the approaches 3294 ft.). The Panama-Pacific Exposition tract is to be one and one-half blocks north of the tunnel end and it is expected that this tunnel will make a great improvement in the transportation facilities of the exposition.



A Proposed Highway Tunnel in San Francisco

A cross-section of a proposed tunnel is shown in the accompanying figure. A separate compartment has been provided for each kind of traffic in each direction. The excavation would be in rock with structural-steel timbering and a concrete lining. In the wagon-traffic compartments it is proposed to use a special roadway with a wood-block foothold for animals and steel ways for the wheels. The tunnel would be built on a grade of very nearly 4%, giving a difference of elevation of 130 ft. between the two ends, which is expected to create a natural ventilating draft. The surface-car compartments would be lighted by the traction companies, while the vehicle and sidewalk sections would be painted white and electric lighted. The total cost is estimated at $866,100, from which a salvage of rock for concrete amounting to $110,000 is expected, leaving a net estimated cost of $756,100. This cost is to be assessed over two assessment districts, one at the north end of the tunnel having an area of 4,630,000 sq.ft. and a frontage of 71,000 ft. The other assessment area at the south end of the tunnel covers 10,000,000 sq.ft. and has a frontage of 144,000 ft.

Calcium Chloride for Drying the Blast of iron furnaces has been tried in Luxemburg, France, by the Differdange Iron & Steel Works. A description of the process, which is the invention of two French engineers, Felix A. Daubine and Eugene V. Roy, is given in the Engineering and Mining Journal of Mar. 23. The installation consists of three cylindrical towers each divided into ten stories or compartments. Each compartment contains a ring-shaped grate supporting a layer of calcium chloride about 10 in. deep. The air to be dried enters at the bottom of the tower and passes upward through a central conduit, then downward through the layers of calcium chloride and upward again through the annular space surrounding the grates. Finally, it reaches a collecting chamber at the top of the tower, whence it flows down through a lateral conduit to its outlet at the level of the fifth story.

The calcium chloride is arranged with small pieces at the bottom of each layer and larger pieces at the top. Moisture from the air is absorbed by the calcium chloride as it passes through. The absorption of the water, however, evolves heat, but this is carried away by water flowing through coils of pipe buried in the layers of chloride.

Only one of the three towers is used for drying the blast at one time. After six or eight hours' use, the stream of air to be dried is turned into the second tower and the chloride in the first tower is regenerated by a current of hot furnace gas tempered by admixture with air. It is claimed that the regeneration can be completed and the chloride fully restored to its initial condition in about four hours. The hot gas is then shut off and the regenerated chloride is cooled by a current of water flowing through the cooling coils already mentioned in order to bring it again to a temperature suitable for the work of desiccation. Thus while one of the towers is being used for desiccation, the second is employed in regenerating the used chloride and the third is being cooled.

The charge of chloride in each tower was 24 tons, the cross-section of the air or gas current was about 1100 sq.ft., and the refrigerating surface for the water pipe about 1800 sq.ft. The cost of the whole installation is given as about $10,000. It is claimed that only one man is required to operate the towers.

The Structure of the "Titanic"

(With two-page plate)

The sinking of the steamer "Titanic," by collision with an iceberg, has directed the attention not only of marine architects but of the whole engineering world to questions in connection with the design and construction of ship hulls. It has seemed to us worth while at this time, therefore, to lay before our readers the general drawings showing the sectional elevation, deck plans and cross-section of this huge steamer, whose loss ranks as the greatest calamity in the annals of marine engineering. We are indebted for these drawings to our London contemporary, *Engineering*, of May 26, 1911, which at the time the "Titanic" was launched (which was about the time her sister ship, the "Olympic," started on her maiden voyage across the Atlantic), laid before its readers an extended account of these two new vessels, the largest ever built.

We abstract from our contemporary's description the following principal particulars regarding the construction of the vessel. Her length over all was 882 ft. 9 in. The extreme breadth was 92 ft., and the molded depth was 64 ft. 6 in. The total height of the hull and superstructure, however, from the keel to the top of the captain's bridge, was 104 ft. As the vessel had a loaded draft of 34 ft. 6 in., the captain's bridge was about 70 ft. above the water level.

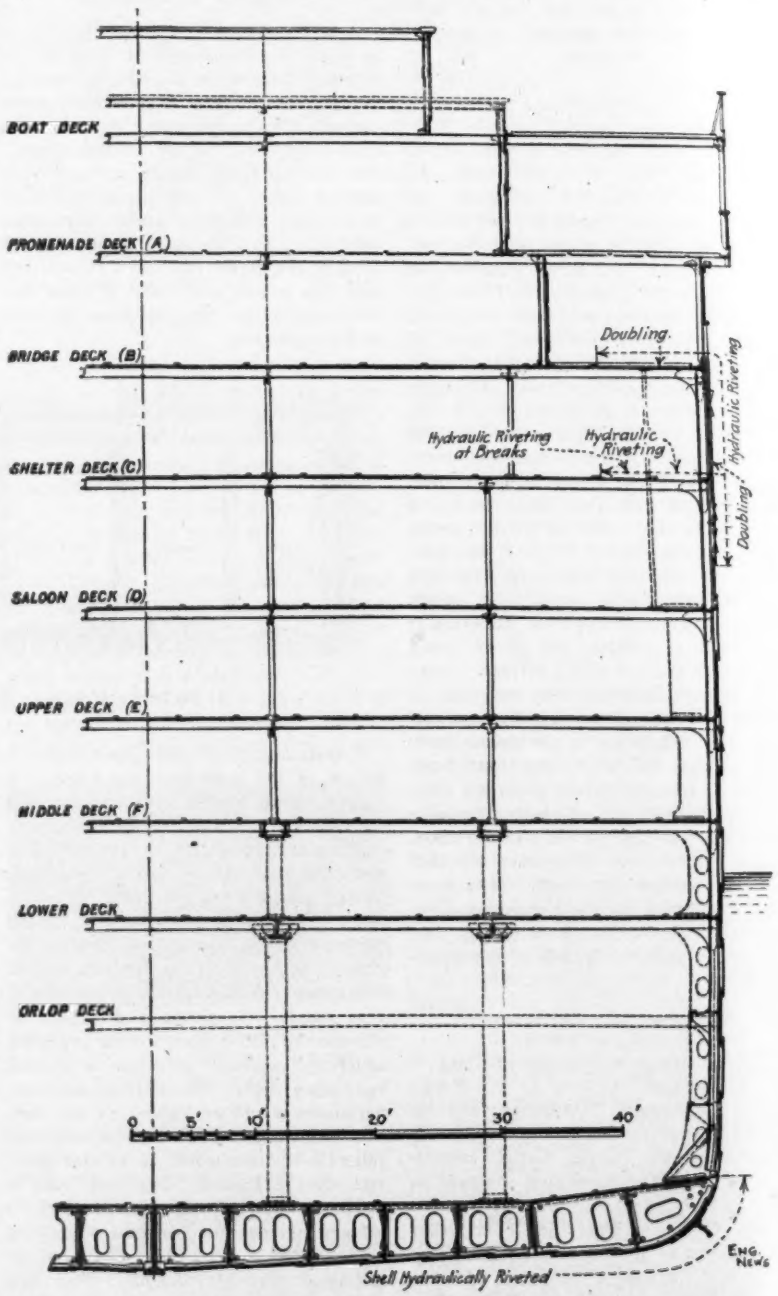
As seen by the sectional elevation, the crow's nest on the foremast, in which the lookouts are located, is about 75 ft. forward of the captain's bridge and some 20 ft. higher. These differences in distance and elevation are comparatively unimportant and it is evident that the officers on the bridge had practically as good an opportunity to discover danger ahead as had the lookouts posted in the crow's nest.

The launching weight of the ship was nearly 27,000 tons; its gross tonnage about 45,000 and when loaded to a draft of 34 ft. 6 in., its displacement was about 60,000 tons. The propelling machinery had an aggregate capacity of about 46,000 hp., and was designed for a continuous sea speed of 21 knots. The vessel was propelled by three screws, the side screws being driven by reciprocating engines, there being one four-cylinder triple-expansion engine on each side-screw shaft. The exhaust from these engines was delivered to a Parsons exhaust steam turbine on the central screw shaft. This turbine was designed to receive steam at 9 lb. absolute pressure, exhausting into a condenser designed to maintain a vacuum of 28½ in. Steam for the engines was furnished by 29 Scotch boilers carrying 215 lb. steam pressure. These boilers were arranged in six separate watertight compartments with five boilers in each, arranged athwart the ship, except in the first boiler room in the bow where, owing to the narrowing of the vessel's width, there was room for only four. The hot gases from the boiler furnaces were delivered into four funnels, the tops of which were 160 ft. above the furnace bars.

Chief interest centers, of course, at this time in the structural details of the hull, and these were described by *Engineering* as follows:

As to the structural details of the hull, the longitudinal section shows that there are 15 transverse water-tight bulkheads, extending from the double bottom to the upper deck at the forward end of the ship, and to the saloon deck at the after end—in both cases well above the load water line. The reciprocating-engine room is the largest compartment in the ship, being about 69 ft. long, while the turbine room is 57 ft. long; the boiler rooms are generally about the same length, and the holds are 50 ft. long. Thus, any two compartments may be flooded without in any way involving the safety of the ship.

The center keelson is 63 in. deep, with lightening holes, and under it there is the flat keel plate, 1½ in. thick. There are four intercostal girders on each side of the center line, with a continuous



HALF CROSS-SECTION OF THE "TITANIC" HULL

margin plate, forming a wing ballast tank at the sides. The frames are spaced 3 ft. apart, except at the ends; the spacing is reduced to 24 in. forward and 27 in. aft. They are generally of channel section, 10 in. deep amidships, and of angle iron and reverse bars at the ends. At intervals there are heavy web frames, and in the machinery space extra strength is provided by increasing the number of these web frames.

As shown, the frames extend from the wing tanks to the bridge deck—a height of 66 ft.—and are connected to the wing brackets by strong plates. The deck beams are 10 in. deep, of channel section to the lower deck; they are spaced to suit the frames, being connected to them by brackets, as shown on the midship section, page 680. Four longitudinal girders extend throughout the whole length of the ship. These are of the plate type, and have deep angle irons.

Special provision is made in the machinery compartment to provide for greater head room; the aggregate sectional area of the girders in the machinery room is equal to the four longitudinal girders in other parts of the ship. The midship section further shows the arrangement of standards in the form of steel circular columns in the holds, engine and boiler spaces. These extend to the lower deck, and above this level there are solid stanchions of less dimensions, but placed at closer intervals. On a line with these stanchions are girders with deck plating of increased thickness, to provide for longitudinal stresses.

The decks are of steel plating, and the shelter and bridge decks are specially strengthened by thicker plates. It will be noted that these deck plates are all scarfed or joggled. In the latter the laps are triple riveted, and at the turn of the bilge, doublings are fitted, as is also the case on the shear strake in the way of the bridge and shelter decks. The cross-section also shows the system whereby the superstructure decks are supported, and, further, the arrangements made so that they may be of greater width than the decks within the molded dimensions of the ship. The upper works are carried on bulb-iron frames on the same lines as the main frames, but spaced wider apart, while heavy brackets are provided at intervals to carry the extension in width of the deck beams.

The vessel had accommodations for 3346 persons, quarters being divided as follows:

First-class passengers	730
Second-class passengers	560
Third-class passengers	1200
Officers and crew	63
Engine-room complement	322
Stewards and victualing department	471
Total	3346

The plan of the boat's deck shows 18 boats and it also shows that a large part of space on this deck was given up to promenades. Separate promenade spaces are shown for first-class passengers, second-class passengers, engineers and officers. It was particularly noted in *Engineering's* description that this promenade was a great attraction to passengers. That journal remarked, "a notable feature is that the boats are so disposed that there is for over 200 ft. of the length of the promenade an unobstructed view over the sea." One of the features on this deck was a gymnasium 46 ft. long and 17 ft. 6 in. wide. There was, therefore, plenty of room on this deck where additional boats might have been stowed.

While direct evidence as to the location where the damage occurred to the vessel's hull and the rate at which the water entered is meager, the indirect testimony seems to make it fairly clear that the ship's hull came in contact with the ice about 20 to 40 ft. back of the stem. All accounts agree that the shock to the hull was comparatively slight. Passengers in remote parts of the vessel hardly felt it at all. That the mass of ice was a huge one, nevertheless, is shown by the fact that the ship crushed into the berg far enough so that pieces of ice were thrown onto the well-deck forward, which is about 38 ft. above the water level. The huge bulk of the ice may be realized by remembering that only a little over 8% of a mass of solid ice floats above the water level. The vessel must have crowded hard against this ice mass, or the pieces of ice could not have been thrown on board.

As to how far back the hull was injured by the ice so as to admit water, evidence is meager. One of the surviving officers had his first warning of the serious nature of the damage by the news that water was coming into the postal room on the lower deck, which is in the third compartment back of the bow, and about at the water line. Immediately afterward firemen came up from their sleeping quarters on the same deck, in the first compartment from the bow, having been driven out by the water. It is therefore evident that the collision opened the front 200 ft. of the hull, except the peak compartment, to the entrance of the sea. The first boiler compartment may also have been injured, as the narrowing of the vessel's lines extends back beyond this compartment.

The second officer of the ship, C. H. Lightoller, however, testified that in his opinion the injury to the hull was confined to the three forward compartments.

The question which is sure to arise as a result of the "Titanic's" loss is what division of a vessel's hull by bulkheads is justified in order to increase its safety and how high in the ship's superstructure should these bulkheads be carried.

In *The Engineer*, of Apr. 19, Prof. J. H. Biles, one of the highest English authorities in marine architecture, discusses the question of safety compartments in ships and explains the origin of the present practice. It appears that in 1890 the British Board of Trade appointed a committee to consider the question of the spacing and the strength of watertight bulkheads. The practice sanctioned by their report is stated by Prof. Biles as follows:

"A vessel is considered to be safe, even in the event of serious damage, if she is able to keep afloat with two adjoining compartments in free communication with the sea. The vessel must, therefore, have efficient transverse watertight bulkheads so fixed that when any two adjoining compartments are opened to the sea, the uppermost watertight deck to which all the bulkheads extend is not brought nearer to the surface of the water than a certain prescribed margin."

Mr. Biles says further: "It is now customary for first-class passenger ships to make it a condition of the design that the spacing of the bulkheads shall fulfill the Bulkhead Committee's requirements when any two compartments are flooded. Responsible persons tend to be satisfied when they have fulfilled either a statutory condition or one which can be definitely interpreted and applied and are disposed to rest content if such a condition is no more than fulfilled. They then allow their vessels to be called unsinkable; but their unsinkability is really only under the conditions named."

Prof. Biles reviews the bulkhead subdivisions adopted in typical large ships. It will be seen from the sectional elevation of the "Titanic" on our inset sheet that the bulkheads aft of bulkhead 2 extended up to deck *E*, which is only a short distance above the normal water line. As the filling of some of the compartments would raise the water line on the hull, it is evident that the margin of safety obtained by bulkhead division is soon exhausted.

It is of much interest to note that the American Line steamer "New York," although built 24 years ago, has all her bulkheads carried up to a deck which is 14 to 15 ft. above the vessel's water line, while some of the vessels built in recent years have their bulkheads carried to a deck only 10 ft. above the water line. The "New York" was designed at a time and under conditions when provision of safety against collision was very much desired by ship owners. Each compartment of the vessel was self-contained.

Quoting further from Prof. Biles' paper, "the boilers of the "New York" were in three separate groups, in three separate compartments, each group having in its own compartment over the boilers all the coal necessary for the voyage. Communication between the boiler rooms was over the top of the bulkheads. There were no doors below through the bulkheads. The passenger compartments were similarly separated. In fact, the vessel up to the bulkhead deck was effectively divided."

The efficiency of this subdivision of the hull was fully tested when the "New York's" sister ship, "The City of Paris," had her engine room flooded through an accident to the machinery and the water found its way into the compartment on the opposite side of the longitudinal bulkhead. The vessel, however, was successfully brought to port.

It is easy to see why this subdivision adopted in the case of the "New York" has not been imitated in later vessels. Quoting still further from Prof. Biles:

> This arrangement of ships has its permanent disadvantages. To fill the space between the main and upper decks in the midlength of the ship with coal was an awful crime from the point of view of the passenger agent. It could have been so well fitted up with state-rooms and the coal be carried down below somewhere. This permanent disability was ever present in the minds of the managers and a contingent escape from sinking at some distant period did not give the necessary solace. In the later ships the subdivision was rearranged. Something apparently more important was favored and a ship with a smaller margin of safety was produced.

Continuing the discussion, Prof. Biles concludes that to increase safety in high-class passenger vessels, the transverse bulkheads should in all cases be carried as high as possible and the deck at the top of the bulkheads should be made effectively water-tight. The difficulties in connection with the propelling machinery, if it is attempted to reduce the size of the separate compartments, and the difficulties with the listing of the vessel introduced by the use of longitudinal bulkheads, make the above suggestion the most promising method for improvement.

It is fair to say, however, that the bulkhead division in modern passenger vessels is intended chiefly to safeguard against damage by collision with other vessels. If one vessel strikes another on the side, as in the case of the "Republic," it will usually penetrate not more than two compartments. If, however, a rapidly moving vessel comes in contact with any large stationary mass such as floating ice or a projecting reef, it is likely to produce leaks along a considerable length of her sides, and admit water into so many compartments at once that the salvage of the vessel will be impossible no matter what her bulkhead subdivision.

Two Rebuilt French Pumping Stations

By C. A. Tupper*

In the city water-works pumping stations of Lyons, France, steam reciprocating pumps have been replaced by electric-driven high-pressure centrifugal pumps—some of them operating against a head of 328 ft.

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The water-works of Lyons, France, supply five areas known from the location of their respective reservoirs, as Bron, St. Clair, Montessuy, Rilleux and La Sarra. The first includes the business district and older section, while the others are outlying. Formerly there were two steam pumping stations, one known as the Grand Camp, serving the Bron district, and the other as the St. Clair supplying all the other reservoirs. Water for all systems is taken from wells on each bank of the River Rhone.

The city's population now exceeds 500,000, and several years ago the water supply was found inadequate; in the year 1909 it was decided to lay larger and stronger mains, to sink 38 new wells, and to increase the pumping capacity to 13,250,000 gal. per day. The St. Clair station was found to be quite obsolete and very wasteful of fuel. As comparatively cheap electric current could be purchased, the use of motor-driven high-pressure centrifugal pumps was decided upon after an investigation of their use for water-works service in Germany and Switzerland, no previous installations having been made in France, except on a small scale and for industrial service.

In the Grand Camp station the engines were comparatively new and economical, so that it was decided to keep these as reserve units and to install the centrifugal pumps between them for normal operation. No reconstruction of the building was necessary, beyond providing light foundation girders and a small transformer annex. The intake, suction and discharge channels remained unaltered so that the installation proceeded without interruption to the service. A plan and elevation of the station may be seen in Fig. 3. This shows that the pressure mains can be fed from either the centrifugal or plunger pumps. The motors used here are a 400-hp. 3500-volt 970-r.p.m. three-phase induction type with wound-rotors and slip rings for external resistance; they are direct-connected to the pumps by flexible couplings. The pumps showed an efficiency of 80.3% at a load of 96 gal. per sec. against 190 ft. head. The motor alone showed an efficiency of 93%, making the combined efficiency 74.6%.

The St. Clair station, shown in Figs. 1 and 2, has been entirely reconstructed, the steam pumps being removed and eight centrifugal units installed, space being reserved for four more.

In this station, particular attention had to be given to the varying conditions of service in the four districts supplied. Rilleux and La Sarra required 16.5 gal. per sec. against 548 ft. head, and 17.3 gal. per sec. against 446 ft., respectively, so that by varying the speed it was possible to use one size and design of pump for both reservoirs. Four units out of an eventual six were installed, any one or all four of which can deliver to either reservoirs, although two pumps are ordinarily assigned to each.

The Rilleux pumps showed a test efficiency of 73.4%, and the La Sarra pumps 74.8%. All four together, delivering to the Rilleux district, showed 74%. This was equivalent to an overall efficiency of 68%, compared with 60% guaranteed.

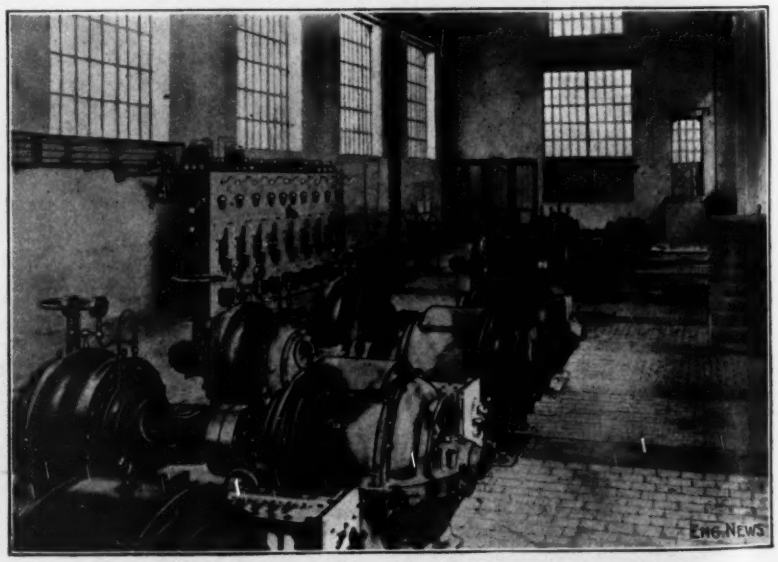


FIG. 1. INTERIOR OF NEW ST. CLAIR PUMPING STATION, LYONS, FRANCE

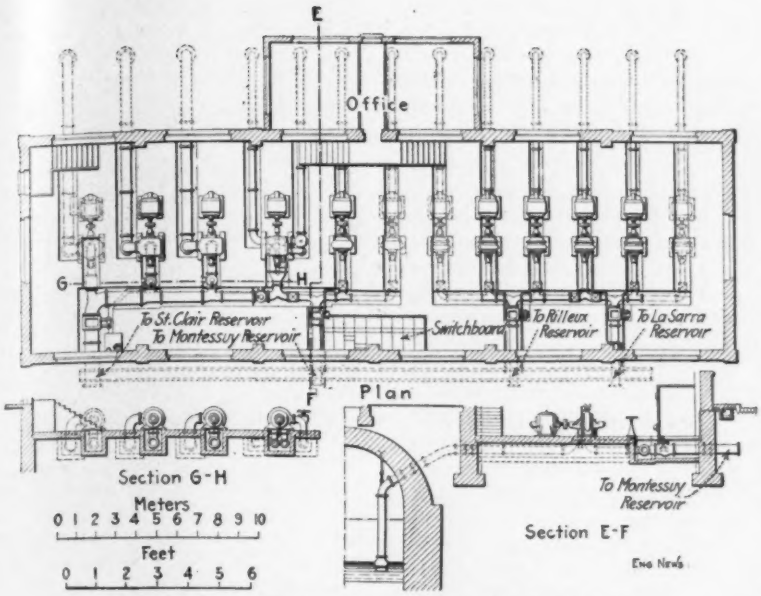


FIG. 2. PLAN AND SECTIONS OF THE NEW ST. CLAIR STATION

St. Clair and Montessuy are to be supplied under quite different conditions. Deliveries of 83 gal. per sec. against 164 ft. and 23 gal. per sec. against 328 ft. being required. The problem of using the same pump for both services was not as easily solved as in the other case, but it was decided best to install pumps capable of supplying either district, particularly for fire protection. The impellers have been arranged so that they can be operated in parallel or in series, supplying 56 gal. per sec. against 164 ft. head or 27 gal. per sec. against 328 ft. Four pumps are provided for these two reservoirs with room for two more if needed. The whole station is practically divided between the two groups of pumping units noted.

The three pumps ordinarily assigned to the St. Clair reservoir showed in a service test an efficiency of 74%, corresponding to an overall efficiency of pumps and motors of 68%, as against 58% guaranteed. The motors for these eight pumps are of the same general design as those in the Grand Camp station, but of 200 hp. at 1450 r.p.m. They showed a test efficiency of 92.5%.

In the St. Clair station the switchboard, with eight motor and one general cut-out panels, is on the floor level at one side; in the Grand Camp station the switchboard is located in a gallery at the end to give an unobstructed view of the pump motors from the switchboard. The controller for each machine is placed at the side of the motor, and the motor circuits are run in floor conduits.

Electric energy is transmitted, from a hydro-electric plant of the Societe Grenoblise de Force et Lumiere, at about 10,000 volts. At the pumping stations it is stepped down to 3500 volts, at which it reaches the motors.

This work was pushed through at the instigation of the Mayor of Lyons, Edouard Heriot, the designs being the work of the city engineers, Dayde, Thomay and Auric. The writer is indebted to the report of the city engineers for test data and plans, and to the pump makers, Sulzer Bros., of Winterthur, Switzerland, for the photograph of Fig. 1. The electrical equipment was made by the Societe Alioth, of Lyons.

The Fall of the 656-ft. Nauen Tower, which was overthrown by the wind, Mar. 30, as noted in our issue of Apr. 11, is described briefly in The Engineer (London), as follows:

The 330-ft. lattice work "Telefunken" tower, at Nauen, was, in the course of last year, raised to twice this height by the stepping of a thinner topmast in a ball-and-socket joint on its cap. The main tower or mast was already stepped in a similar manner on the ground, and mast and topmast were each held up by a system of stays. During the storm on Mar. 30, the tower was blown down.

According to published accounts, the top mast was the first to fall to leeward with a somersault well clear of the buildings below. The 80-ton lower mast then began to rock more violently, and of the three stays that held it the leeward one was the first to break. During a lull in the wind the tower then swayed over to windward, and then swung backward and forward with increasing violence till it fell over to the windward side.

The damage is estimated at several hundred thousand marks. The accident is attributed by experts to faulty design of the structure. The lower half was probably strong enough in itself, but the subsequent addition was too much for it. Differing considerably in diameter, the two parts had oscillation periods of different duration. Varying in intensity as storm gusts do, different vibrations were set up in the two parts of the tower, and these sometimes coincided with and sometimes were opposed to each other. The resonance effects of the upper and lower parts acting together on the weak point, viz., the step of the latter, are considered to have been the immediate cause of the catastrophe.

The Telefunken Co. hopes to rebuild the tower in the course of this year. Since the tower had not been put to work, no derangement of service is caused by the accident.

An account of this accident in the "Zeitschrift des Vereines deutscher Ingenieure" states that the three stays holding the lower mast were attached at a point three-fourths its height above the ground. The upper mast had nine stays, of which the three lowest were attached close to its base. Both towers were electrically isolated from the ground and from each other by glass insulating pieces. The fall of the upper mast, according to the "Zeitschrift," resulted from the failure of one of its stays on the windward side. In its fall the upper mast severed the leeward stay of the lower mast. The pressure of the wind kept the lower mast up for about seven minutes, until during a lull the weight of the two remaining stays pulled it over to windward.

A Tunnel 5.32 Miles Long has just been started in Switzerland. It passes through the Grenchenberg, and will be 8565 m. or 28,093 ft. long between portals. Driving was started in February and by the end of March 1070 ft. had been driven (both headings).

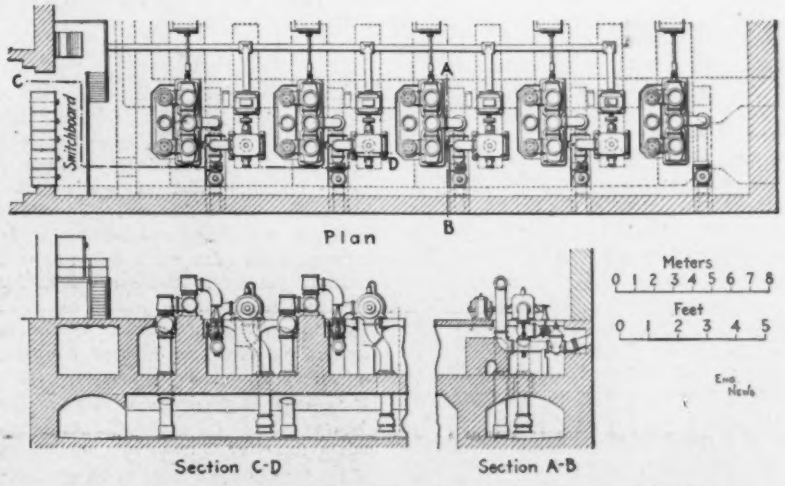


FIG. 3. PLAN AND SECTIONS OF THE GRAND CAMP PUMPING STATION, LYONS FRANCE

Water Waste Surveys at Chicago

Excessive use and waste of water is a common condition in this country and has reached a specially serious degree in Chicago. The popular idea of meeting any scarcity of supply is to increase the supply, but this involves heavy expenditure of public funds. The more economical method is to check waste and reckless use, partly by the introduction of meters and partly by the repair of defective pipes and plumbing. This latter method is being followed out systematically in Chicago, and a thorough survey made to detect waste and leakage. This survey and its results are here described in detail. It is believed that in this way the supply may be so conserved that the proposed expenditure of large sums for additional pumping engines may bepostponed for a considerable time.

The enormous waste of water in Chicago is a problem of long standing, and is one reason why in certain districts the supply is insufficient for fire protection and is available only on the lower floors of buildings. The city is making great expenditures in extending the water-supply system and increasing the pumping capacity, but the proportion of waste is so great as to prevent these improvements to a large extent from giving their full and proper effect. The reduction of waste is of the utmost importance in regard to both the economy and the efficiency of the water-supply system, and the city is working toward this end, partly by the introduction of meters (which as yet have been applied to but a small percentage of the services) and partly by means of a water survey to detect and locate losses by leakage and waste. Additional capacity of supply is being provided also, and it is estimated that the expenditure for this purpose during 1912 will be $3,568,500, distributed substantially as follows: improvements to pumping stations, $1,860,500; improvements to distribution stations (or booster stations), $28,000; pipe extension, $430,000; feeder mains, $1,250,000. At the present time the water supply in certain districts is insufficient for fire protection and is available only on the lower floors of buildings. These conditions are shown by the map, Fig. 1, but it is considered that by reduction of waste in different ways the present supply may be made sufficient and the contemplated construction of new pumping stations may be postponed for a considerable length of time.

General Water-supply Conditions, 1911

According to the annual report of the city engineer, the total pumpage during 1911 was 185,176,300,000 gal., or an average of 507,332,000 gal. per day. This represents the total plunger displacement of all pumping engines in service during 1911. The net amount of water actually delivered into the mains at the stations is less than the amount shown by the plunger displacement, on account of the slippage of the pumps. The average slippage of all pumps in service is estimated to be 15%, and upon this basis the pumps delivered into the mains at the pumping stations only 85% of the amount of water represented by the total plunger displacement. The total average daily supply for the year 1911 was, therefore, 413,232,000 gal., and the average daily supply per capita (based upon this figure and an estimated population of 2,259,000) was 190.8 gal.

At the close of 1911 the Chicago waterworks system consisted of nine large pumping stations and five small stations, 49.42 miles of water tunnels (from 5 ft. diameter to 14 ft. largest dimension horse-shoe section), 2362.5 miles of water mains, 20,485 valves, and 24,897 hydrants. The number of services in use, based upon the number of consumers, was 258,860. The number of meters in use was 15,762, which is 6% of the total number of services. The total revenue from the sale of water during 1911 was $5,887,637. The revenue received for metered water was $2,556,594, and for water paid for by frontage $3,331,042. The metered consumer, therefore, pays 43.3%, and the frontage consumer 56.7% of the total revenue.

Water Survey

The water survey mentioned above is in the hands of the Water Survey Division of the Bureau of Engineering, and the report for 1911 contains some very interesting information as to the defective conditions and the remedial methods in regard to leakage and waste. While this report has been made and approved, it would not be available normally for some months, when the printed report of the Department of Public Works is issued. We have been favored, however, with facilities for examining the report in advance of publication, through the courtesy of Mr. Ericson, city engineer, and in this way we are able to present the the following information to our readers. The report in question was prepared by T. C. Phillips, engineer in charge of water surveys. The water survey includes house-to-house inspection, district surveys, flow-tests of trunk or large feeder mains, meter and pump-slip tests, and leakage tests of mains in the streets before paving is laid. The Water Survey Division also prepares lists of premises where the installation of meters is desirable, has charge of the repair and operation of all valves of 16 in. and over, and in general looks after the conservation of the water supply. The appropriations for its work for 1911 and 1912 were, respectively, $154,544 and $163,147.

The total daily saving of water during 1911, due to this work, was about 15,000,000 gal. by eliminating plumbing leakage (in 35 districts) and 4,000,000 gal. by eliminating underground loss in the streets. The value of this saving of 19,000,000 gal. daily, at 3c. per 1000 gal., amounts to $208,050 per annum. The total saving would have been greater but for the low-pressure conditions of the water supply during 1911. The surveys covered 12 sq.miles of densely populated city area, and the survey work in 15 districts (averaging 160 acres) was practically completed. The city is dided into 103 districts for this work. The expenses for eliminating losses by plumb- 178 and $9121, respectively. In addition, miscellaneous routine work cost $18,956, making the total expenditure of the water-survey department $111,255 for the year.

Inspection versus Metering

It has been found profitable to make house-to-house inspection in practically all the territory surveyed, but it is not necessary in premises that are metered or in a few high-class residence sections where the plumbing is generally kept in good repair or where meters are used. The saving is made mainly in districts where plumbing conditions are inferior. Two visits to each place by the inspector are generally sufficient, but in a few places additional visits are required before the plumbing repair is made properly. On re-inspection, one man will visit and report on 10 to 20 places daily, but the first inspection takes a little more time. The inspectors are competent and experienced plumbers, and are paid $143 per month. During 1911, in 37 districts (12 sq.miles), there were 72,345 inspections made, covering 583,142 fixtures; the number of leaks detected was 41,390 above ground and 21,663 underground. The total measured leakage amounted to 18,364,800 gal. per day and the cost of inspection was about $66,874, including the tests of each district by the pitometer field party.

The responsibility of keeping the plumbing in good order and free from

leaks falls upon the owner, tenant or consumer. In case of failure to stop leakage the commissioner of public works may shut off the water supply (after due notice), but the ordinance covering this point is being violated constantly. Plumbing leaks may be eliminated either by house-to-house inspection or by metering. The latter is the more efficient and is permanent. The efficiency of the former depends almost directly upon the inspector. The permanency of plumbing repair is indefinite; the plumbing may be made as good as new or the repairs may be only temporary. The permanency is much greater in districts having first-class fittings and fixtures. In older sections and where the plumbing is poor, frequent inspection is necessary to maintain good conditions and prevent continuous waste.

Where water is wasted by the improper adjustment or defective condition of water-regulating devices, the permanent saving due to inspection is probably very small, as the consumer is likely to resume the wasteful conditions almost immediately. Hopper closets, automatically controlled closets and urinals, hydraulic motors and pumps, and flushing appliances of all kinds belong to the class of most wasteful and extravagant appliances. The annual revenue derived by the city from the use of these appliances is no doubt much less than the cost of the water supplied. A meter was applied to a hopper closet found to be running continually and showed a loss of 10,000 gal. daily. At 3c. per 1000 gal. for cost of supply, this represents $110 per year, while the consumer in this case is charged only $3 or $3.50 per year. It is estimated that there are over 60,000 long-hopper closets in the city, and that the total number of the other appliances is exceedingly large. Inspection does not secure permanent results with these appliances; they should be controlled by metering and the city thus relieved from regulating them.

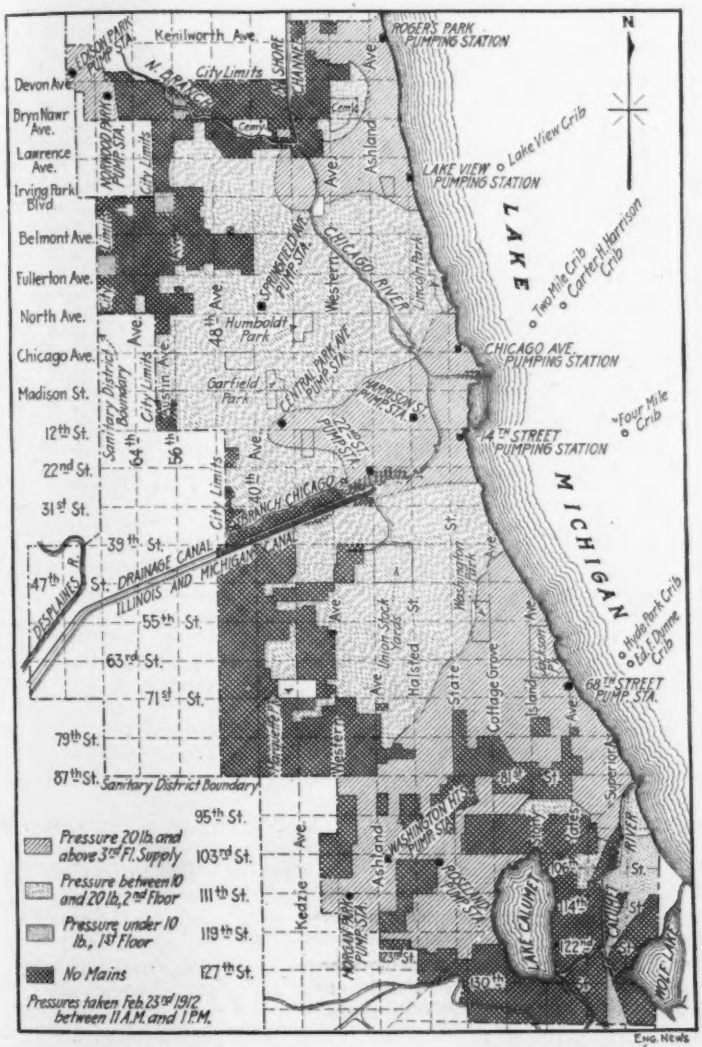


FIG. 1. MAP OF CHICAGO SHOWING THE DISTRICTS OF HIGH AND LOW PRESSURE OF THE WATER SUPPLY SYSTEM

LOSS DUE TO HOPPER CLOSETS

In view of the great number and general deficiency of long-hopper closets and the tendency to leave water running in them during cold weather, a special survey was made as to the loss due to these appliances during cold weather. The closets of this type have the flushing water delivered directly from the main instead of from a flush tank, and are found in the poorer classes of buildings. They are usually accompanied by poor plumbing conditions. There is a common belief among consumers that to let the water run promotes sanitary efficiency; this is a fallacy which is difficult to overcome and is costly to the water department. For this survey two districts were selected, one on the west side (No. 62) and one on the south side (No. 5). In No. 62 (160 acres) there were 17,500 people and 1198 long-hopper closets; No. 5 (258 acres) had 20,000 people and 1269 hopper closets. The results are shown in Table I and in the diagrams on Fig. 2, in which each curve represents a 7-day test. The first and second tests were made before and after the inspection and repair of the plumbing. The third was made during the severe cold weather of January, 1912. It will be noted that the normal drop during the night does not occur in this test, indicating that the water was allowed to run continuously in the hopper closets. In district No. 62, the greatest demand in cold weather seemed to follow the area having the greatest density of hoppers, but in No. 5 (where the consumption was doubled) this could not be determined. In a house-to-house inspection of 2471 places in No. 5, there were found 1738 leaky fixtures and 1565 underground leaks inside premises.

This excessive consumption comes usually at a time of generally increased demand, and the fire risk is consequently increased, while there is inconvenience and risk caused by the low pressure. It is considered doubtful if this source of waste can be regulated without the use of meters.

TABLE I. INCREASE IN CONSUMPTION DUE TO LETTING WATER RUN DURING COLD WEATHER

	Average daily flow; gals. Dist. No. 62	Dist. No. 5
1. Before inspection of plumbing	2,066,000	3,256,000
2. After repair of plumbing (35° F.)	1,558,000	2,531,000
3. During cold weather (48° F.) (19° F.)	2,272,000	5,108,000
Increase of 3 over 2	46%	100%
Ave. pressure, second test	22 lb.	24 lb.
Ave. pressure; third test	12½ lb.	15 to 20 lb.
Saving by repairing faulty fixtures, gals. per day	508,000	727,000

METERS

At the end of 1911, there were 258,860 services and 15,762 meters (or 6% of the services). About 25½% of the net pumpage is passed through meters. The report states that the principal advantage of a meter is that it is undoubtedly the best detector and inspector of leakage and waste beyond the meter, and it is perhaps the fairest regulation of revenue. There is no question as to the great benefit to be derived from an extensive system of metering, and it is the policy of the city to increase the use of meters. We may note here that it is difficult to show the results thus far obtained, since the meters are scattered throughout the various districts into which the city is divided for water-survey purposes. If a large number of meters could be concentrated in some one district, a survey of that district (such as has been noted above) would show the comparative results with and without metering. We understand that it is proposed to carry this plan into effect. Meters would eliminate the water consumption due to plumbing leakage, which is 20% of total pumpage (as shown in Fig. 3), and a large portion of the underground leakage and waste (50%), except that due to leakage in the street; they would regulate free service (5%) and the actual usage by frontage (19½%).

It is shown in the report that the benefit of an extensive system of metering should be considered from the standpoint of reducing the cost of water both by curtailing the losses of supply and by keeping down the cost due to investment of capital. A comparison of the lake cities of Cleveland and Milwaukee (practically all metered) with Chicago (having a small percentage of services metered), with conditions of consumption similar, shows a daily average consumption of about 100 gal. per capita for the first two cities as against 200 gal. for Chicago. It is fair to conclude that both Cleveland and Milwaukee would have had a per capita consumption as large as that of Chicago if meters had not been generally introduced.

UNDERGROUND LEAKAGE AND WASTAGE

On this subject we quote the report practically in full, since it is of particular importance. In determining what becomes of the water supply and analyzing its factors of usage, waste and leakage, we must recognize a loss due to leakage from mains and service pipes in the street. This loss is due to joint leakage and to damage to pipes from electrolysis, and also to the various causes which rupture and destroy service pipes. The proportion of the supply delivered to the mains which is lost through underground leakage in the streets is difficult to determine. The survey data, from which underground determinations were made, was based upon the minimum night rate of flow of the second survey, which is always made after repairing the plumbing. An average allowance of 25% of this minimum night rate was made for usage; but actually this may vary from 10% to 50 or 70%, depending upon the class of consumers in the district. After deducting 25% for legitimate usage, the remaining 75% was treated as underground leakage, and "an unaccounted for loss" which we will refer to as "wastage" until it can be accounted for with more certainty.

Unfortunately, in Chicago the stop-cock on the service pipe at the curb has fallen into disuse to such an extent that the services in use cannot be shut off during a test. As this condition is more or less universal and repair requires considerable time as well as expense, accurate determinations for leakage in the street have been made only in a few cases where the street or section indicated excessive flow. A few cities have obtained good results by driving a rod to the main or service and detecting leakage by means of an aquaphone or similar device, or by digging test holes. These methods, however, are slow, tedious and expensive where paving foundations are heavy as in Chicago. Shutting off the services at the stop-cock at the curb and measuring the flow into a street with a pitometer or a meter is positive and gives an absolute determination of the amount of leakage in the street. In a number of instances mains with a few live services were tested in this manner. In such cases absolute results were obtained and permanent repairs made.

As an illustration of the difficulty of determining the proportion of the minimum flow into a street during the small hours of night that is due to leakage from the mains and services under the pavement, the results are given of a test made in Troy St. Two tests at night showed a rate of 420,000 gal. per day for the four blocks under test. Then a test was made for each block, and showed rates varying from 55,716 to 127,150 gal. per day per block. A house-to-house inspection showed a total measured fixture leakage of 29,760 gal. per day for the four blocks. This test shows plainly that the minimum night rate flowing into a street, with services open and even with plumbing in buildings along the street in good repair, cannot be totally charged to underground leakage in the streets.

Another case of underground leakage was discovered in Chicago Ave., where the minimum night rate was found to be 280,000 gal. per day. The length of street under test was 1335 ft., and had 54 live services connected to a 16-in. main. The 54 live services were shut off at the curb and the flow then registered 245,000 gal. per day. This was plainly underground leakage in the street. Consequently, the 16-in. main, which lay in loose rock under a heavy concrete street-

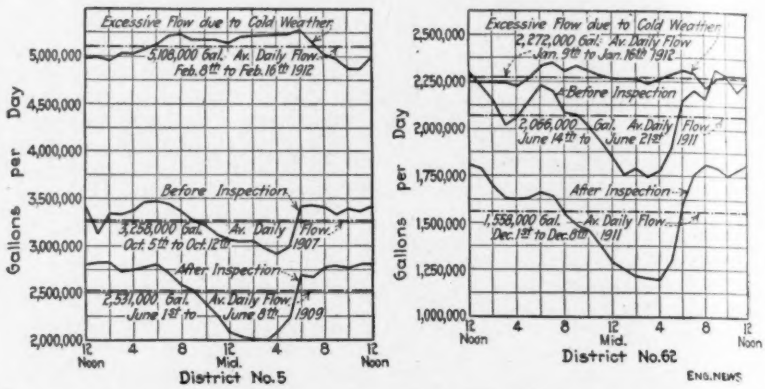

Excessive Flow due to Cold Weather
5,106,000 Gal. Av. Daily Flow Feb. 8th to Feb. 16th 1912
Before Inspection
3,258,000 Gal. Av. Daily Flow Oct. 5th to Oct. 12th 1907
After Inspection
2,531,000 Gal. Av. Daily Flow June 1st to June 8th 1909
Gallons per Day
5,000,000
4,500,000
4,000,000
3,500,000
3,000,000
2,500,000
2,000,000
12 Noon 4 8 12 Mid. 4 8 12 Noon
District No.5
Excessive Flow due to Cold Weather
2,272,000 Gal. Av. Daily Flow Jan. 9th to Jan. 16th 1912
A Before Inspection
2,066,000 Gal. Av. Daily Flow June 14th to June 21st 1911
After Inspection
1,558,000 Gal. Av. Daily Flow Dec. 1st to Dec. 8th 1911
Gallons per Day
2,500,000
2,250,000
2,000,000
1,750,000
1,500,000
1,250,000
1,000,000
12 Noon 4 8 12 Mid. 4 8 12 Noon
District No.62
ENG. NEWS

FIG. 2. DIAGRAM OF WATER CONSUMPTION IN TWO DISTRICTS OF CHICAGO

This shows (1) reduction in water consumption by inspection and repair of plumbing, and (2) increase in consumption due to letting water run from fixtures (especially long-hopper closets) during severe cold weather.

TABLE II. DISTRIBUTION OF WATER SUPPLY IN CHICAGO: 1911

		Million-gal. per year	Ave. gal. per 24 hr.	Per Cent.	Remarks
1.	Actual usage of frontage	30,654.88	83,985,982	19.48	1 – 7 – (2, 3, 4, 5 and 6)
2.	Metered supply	36,522.77	100,062,400	23.20	Based on revenue and flat rate of 7c. per 1000 gal.
3.	Slip of meters	3,652.27	10,006,240	2.32	Estimated at 10%
4.	Free service	7,869.99	21,561,625	5.00	Estimated
5.	Plumbing leakage	31,479.97	86,246,490	20.00	Based on surveys
6.	Underground leakage and wastage	47,219.96	129,369,748	30.00	Based on surveys
7.	Total net pumpage	157,399.85	431,232,494	100.00	Actual water delivered to mains at pumping stations
8.	Total pumpage (plunger displacement)	185,176.30	507,332,328	.....	Includes 15% allowance for slippage of pumps

car track foundation, was stripped. In the repair work, 15 leaky service pipes were repaired, 5 leaky joints were calked, 9 drive taps were replaced with screw taps, one new tap was made, and two taps with old services were abandoned. The repair work stopped all leakage and cost $639. A saving of 245,000 gal. per day, at 3c. per 1000 gal., amounts to $2683.

These two cases are examples of other tests made. They show that the minimum night rate on a street may be due entirely to wastage inside premises, or to both wastage and underground leakage. During the past year the city laid over 120 miles of pavement. The water mains were tested before the pavement was laid to ascertain if the main was free from leakage in the street and in good condition. This is called street-improvement work, and is carried on separately from the district surveys. The flow into a street was measured at night and included the waste and leakage inside premises in addition to the underground leakage in the street. These two elements could not be separated on account of the condition of the stop-cocks that control the service pipes.

Under the old ordinance the property owner was required to keep the curb cock and buffalo box in repair. Under the new ordinance this is not provided for. The result of the failure in keeping the stop-cock in repair has been costly. Someone must pay for the repair when it becomes necessary to use the stop-cock for any purpose. The practical value of the curb cock in good repair as a means of attacking the underground losses of water in the street has been forcefully demonstrated in other American cities. The estimates for underground leakage in Chicago, as stated previously, are based upon the rate of flow during the small hours of the night. The dividing line between the underground leakage in the street and usage and waste inside premises is at the stop-cock at the buffalo box.

It is doubtful if permanent results in eliminating underground leakage in the street will be accomplished over the general area of the city until the flow into a main can be measured with all the service pipes shut off at the curb. Such a test shows definitely the amount of underground leakage in the street main and gives an accurate basis for determining whether the main in the street shall be overhauled. It is estimated that over 50% of the buffalo boxes cannot be operated or have disappeared. The repair of the buffalo boxes and curb cocks would probably cost about $6 each, if done in quantity. The work should proceed along with metering and street-improvement work as a part of the maintenance of the department. If undertaken systematically the work could be done at a rate that would cover the city in five to eight years, without any considerably extra burden of expense.

What Becomes of the Water Supply

The water that is delivered into the mains at the pumping station may be divided into "water that is used" and "water that is wasted, lost and unaccounted for." Table II shows what becomes of the water supply. These determinations are based upon data and information compiled by the water department, and from district surveys made in older and densely populated sections, including house-to-house plumbing inspections in a few outlying districts. The same information (plotted for each month) is given graphically in Fig. 3.

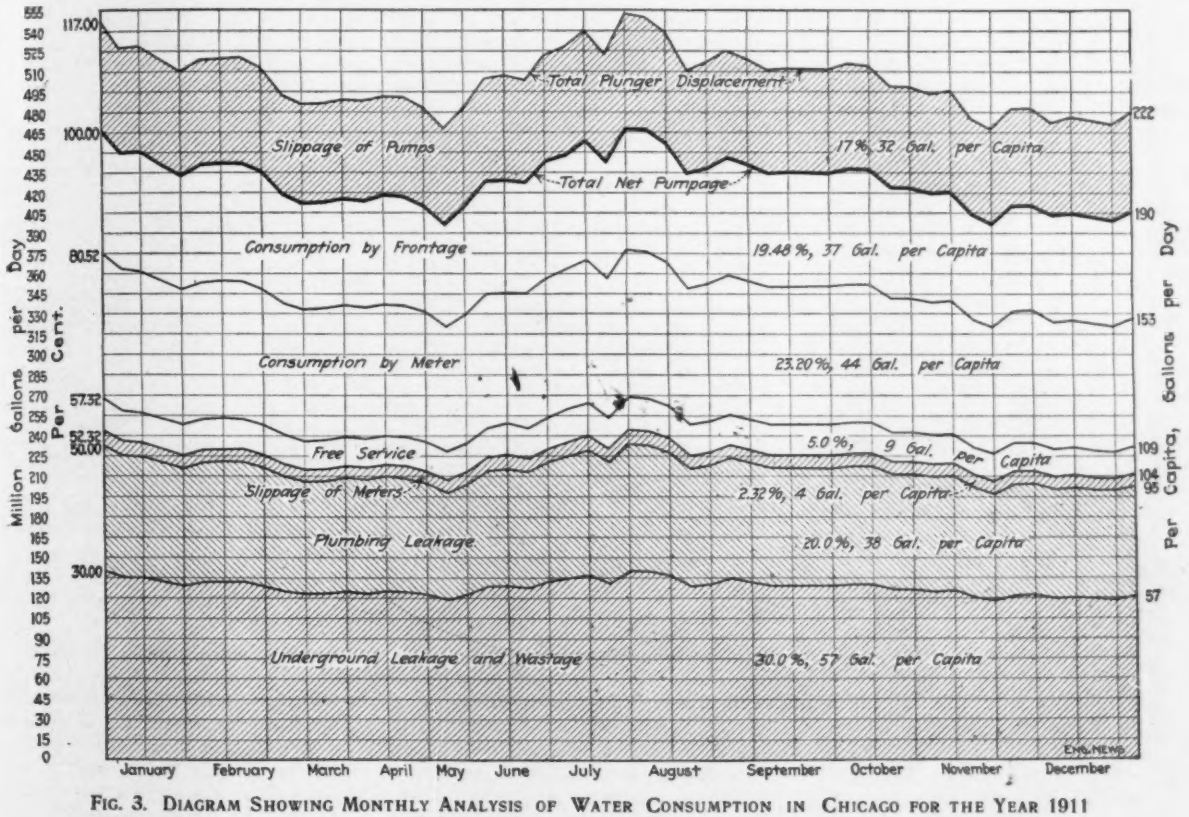


Fig. 3. Diagram Showing Monthly Analysis of Water Consumption in Chicago for the Year 1911

The four principal items in the table are plumbing leakage, underground leakage and wastage, usage by frontage, and water registered by meters. These embrace over 90% of the total supply. The amount of "free service" given as 5% may be much more or considerably less than this amount, as there is very little information upon which to base an estimate of the amount of this usage. The plumbing leakage was determined by measuring the supply into a district before and after the plumbing was repaired. The underground leakage in the street cannot be determined separately from the losses due to willful wastage and neglect

inside premises, on account of the condition of the stop-cock at the curb which controls the service pipe, as already noted. The night flow into the districts surveyed is generally very high and excessive, which is indicative of underground leakage and waste. A large number of tests have been analyzed and they tend to emphasize the fact that high night flows are due largely to causes that originate inside premises and are not wholly chargeable to underground leakage in the street. The amount of metered water was obtained from the total registration of meters. The slip of meters was estimated at 10%. There are no data at hand showing directly the amount of water used by frontage consumers. This was obtained indirectly after values were obtained for the other items in the table.

From the figures given, it can be seen that the frontage consumer pays 56.7% of the total revenue and receives 39.41% of the total net pumpage; and that the meter-rate consumer pays 43.3% of the revenue and receives 25.52% of the pumpage. The table shows that 25.52% of the water delivered into the mains is water passing through meters, and that 24.48% of the water delivered is water used by frontage and by free service. These two items represent fairly the water actually used, and amount to about 50% of the water delivered into the mains at the stations. Not all this water, however, is used in producing revenue, as part of it (5%) is for free service, and another part (2.32%) is lost in meter slippage. In other words, 42.68% of the net pumpage produces the entire revenue and 57.32% produces no revenue.

Under the ordinance the meter rate for water is approximately 7c. per 1000 gal. The consumer who pays by meter receives some water which he does not pay for on account of the slippage of meters (estimated at 10%). The metered consumers, therefore, pay only 6.3c. per 1000 gal. for their actual supply. The table shows that the frontage consumer receives 39.48% of the net pumpage, about one-half of which he uses and the other half he wastes through bad plumbing. For the half he uses he pays 10.23c. per 1000 gal. The city receives about 5.05c. per 1000 gal. for the 39.48% of the total net pumpage it delivers to the frontage consumers.

If by means of meters, or by other means, the water that is now supplied to frontage consumers can be reduced 50%, the city would have this 20% of its entire supply for delivery elsewhere. It could be used for the 20% reserve supply that is needed. It would delay the day of construction for 100,000,000-gal. pumping stations in contemplation for the future and thereby save the city a considerable sum of money.

New Terminals for the Brooklyn Bridge

The approaching completion in New York City of the Center St. Loop Subway, whose southern portion passes under the large new Municipal Building, brings into prospect a solution of the Brooklyn Bridge terminal problem, after more than 20 years of fruitless agitation and planning. The tracks of this subway will connect with the tracks of the elevated railway crossing the Brooklyn Bridge, leading on the Brooklyn side to the extensive elevated-railway network of the Brooklyn Rapid Transit Co. At its other end the Loop Subway connects with the elevated tracks of the Williamsburg Bridge; this, and an intermediate connection with the tracks of the Manhattan Bridge, enables the Brooklyn system to bring passengers into Manhattan over three routes, and on both directions of running. When this system is in full operation most of the trains now crossing the Brooklyn Bridge will, instead of discharging their passengers in the stub terminal at the Manhattan end of the Brooklyn Bridge, continue to the Municipal Bldg. station and beyond to other stations on the Center St. Loop, and thus distribute their traffic (both outgoing and incoming), instead of concentrating it at a single center of congestion.

The Brooklyn Bridge terminals present the problem of handling a remarkably heavy traffic in limited space. The following figures, from a recent bridge-traffic count (Dec. 28, 1911) are illuminating:

TRAFFIC CROSSING BROOKLYN BRIDGE IN 24 HOURS

Elevated ry. cars*	7,913	
Surface ry. cars†	8,500	
Road vehicles	3,568	
Total vehicles		19,981
Elevated ry. passengers	217,110	
Surface ry. passengers	114,049	
Road vehicle passengers	5,404	
Pedestrians	5,714	
Total persons		342,277

TRAFFIC THROUGH BROOKLYN TERMINAL IN 24 HOURS

Add to above the elevated ry. cars that turn at bridge or go to Fulton Ferry, about	8,000
Total vehicles	[illegible]

*Rate of running elevated trains in one direction across bridge during rush hours, 60 to 64 trains per hour (one track).

†Rate of running surface cars across bridge in one direction during rush hours, 360 cars per hour (one track).

New Manhattan Terminal

The work of connecting the Loop Subway tracks with the Brooklyn Bridge tracks gives occasion for a general reconstruction of the bridge terminal opposite City Hall, on Park Row. A few weeks ago (ENGINEERING NEWS, Feb. 22, 1912) we showed the architectural features of this proposed new terminal, which will form a conspicuous part of the City Hall surroundings. The new terminal, however, is considerably more important and interesting as a public utility than as an architectural feature. The picture reproduced herewith as Fig. 1 is a sectional perspective prepared in the office of the Department of Bridges, New York City, to show the arrangement of the new terminal with respect to surrounding structures. The officials of the department rightly emphasize the fact that the arrangement shown will create a transportation center equalled by few others in the world for number of transit lines and concentration of transportation facilities. To quote briefly from a description prepared by the bridge department.

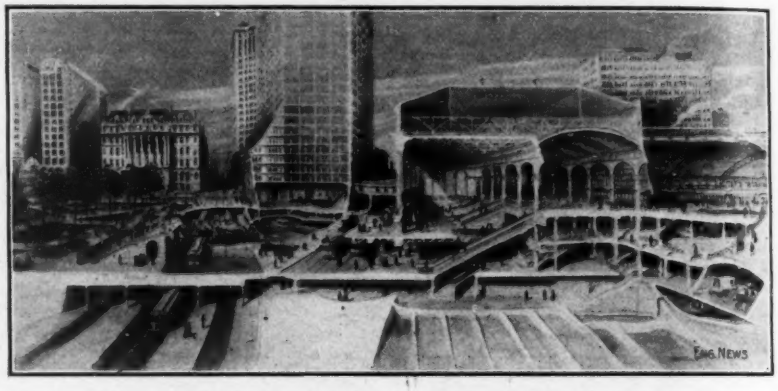


FIG. 1. PERSPECTIVE SECTION OF NEW (PROJECTED) TERMINAL AT MANHATTAN END OF BROOKLYN BRIDGE, NEW YORK CITY
(Connection of Brooklyn Bridge tracks with Center Street Loop Subway.)

The picture shows the present Interborough Rapid Transit subway on the left, and on top the surface cars of the New York Railways Co. on Center St. Under the Municipal Building (in the center) is shown the subway connection with the Brooklyn Bridge via the Center St. Loop, and above the tracks a mezzanine floor for distributing the passengers to the trains and a connection with the mezzanine floor of the present subway. Over Park Row is shown the connecting bridge between the Municipal Building and the headhouse of the

Brooklyn Bridge terminal. Back of this is the Third Avenue Elevated Railway terminal, not shown.

To the right is the section of the proposed new bridge headhouse and trainshed. On the lower floor is an underground passage connecting the bridge promenade with the present subway by several stairways and also connecting with North William St. In the lower right-hand corner is shown a train on the bridge-subway connection, being the continuation of the tracks shown under the Municipal Building. Directly over this is the south driveway and on the same floor the bridge promenade, from which stairways ascend to the mezzanine floor for the elevated trains and to the respective trolley loops. The mezzanine floor may be approached by a bridge over Park Row. The incline in the foreground is an escalator which conveys people from the street level directly to the concourse floor; this escalator is located in the middle of the building.

The bridge-terminal tracks will be about equal in capacity to the existing terminal, but will not have to accommodate as many trains since the larger number will go by way of the connection to the Loop Subway without stopping at the bridge terminal. The terminal tracks are arranged with incoming and outgoing platforms separated, and the stairways leading from these platforms to the street are kept separate throughout so that the streams of people moving in opposite direction need not conflict.

Concerning the arrangement of the tracks at the junction of the new connection from the Loop Subway with the existing bridge tracks, it need only be said that these tracks (two) rise from the subway on a grade of slightly over 5%, on such alignment that they lead straight into the (two) bridge tracks. The two tracks from the junction to the bridge terminal lie outside of the depressed tracks, being entered by turnouts at the junction. The junction lies just west of the Franklin Square crossing of the bridge.

In connection with the characterization of the new terminal as an important traffic center, it is to be remembered that the new Broadway subway now under construction (and according to latest projects to be operated by the Brooklyn Rapid Transit Co., from a tunnel to be constructed at the Battery, via Broadway and Seventh Ave. to 59th St. and then east to and across the Blackwell's Island Bridge) lies only one block west of the Municipal Bldg., and the Sixth Ave. elevated railway is two blocks beyond Broadway. Thus, five important rapid-transit lines are here within 2000 ft. of each other, and the incoming traffic of the Brooklyn Bridge is brought within reach of four of them and leads directly upon the fifth, the new Loop Subway.

Reconstruction of Brooklyn Terminal

Extensive improvement and reconstruction of the Brooklyn terminal of the bridge is also contemplated, and we reproduce herewith the perspective, Fig. 2, showing the projected new arrangement and structure. The station shown at the end of the bridge is practically on the location of the present terminal structure, but greatly altered and enlarged as to train capacity. The trackage forward of this is rearranged to remove the tracks from Fulton St. and thereby permit the creation of a large open plaza to relieve street congestion and simplify the surface-car operation. This portion involves changes in the lines of the Brooklyn Rapid Transit Co. The changes in the structure beyond would be made by the city.

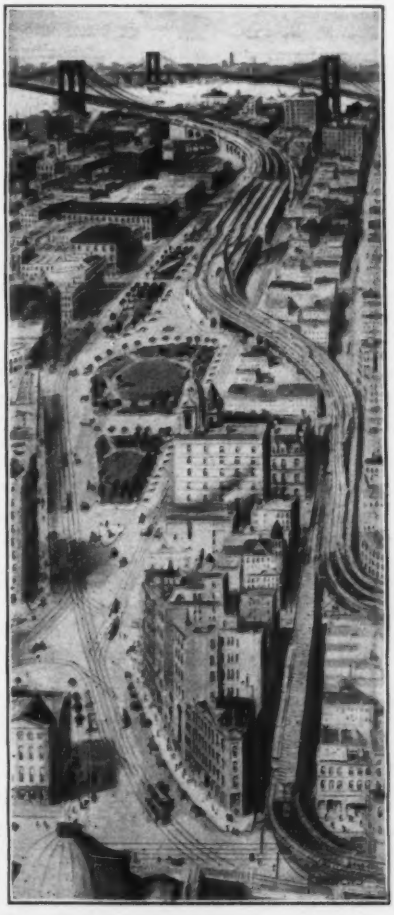


Fig. 2. General View of Proposed Reconstruction of the Brooklyn Terminal of Brooklyn Bridge

The most apparent change of the present terminal or station is the removal of the large station house with its complicated provisions for routing of trains and passengers. A loop of the elevated track passing easterly from the present bridge terminal through Sands St. and High St. would be removed, and all trains crossing the bridge would pass through the station shown. The trackage here is arranged in simple manner with sheds over the station platforms, but no general inclosure. Crossovers between the tracks will permit of operation of trains in any desired manner and will make provision for running a shuttle service over the bridge, or returning all elevated trains without crossing the bridge, as might be necessary in order to continue service when a track is blocked by an accident. The station will at the same time provide improved facilities for the trolley-car service over the bridge, accommodating these cars on a mezzanine floor under the floor devoted to the elevated tracks. The team driveways at the outer edges of the structure are also rearranged in their approaches for greater efficiency and convenience.

As may be seen in the distance, an elevated track turns out from the bridge station to the left, leading to Fulton Ferry. The connection of this track with the bridge tracks is also altered. Under the new arrangement the trains for Fulton Ferry would pass into the station from the bridge and then out via a depressed track to the Fulton Ferry elevated structure. In this way the handling of trains is unified and systematized in such a way as to make operation most regular and free of confusion

The features of this arrangement involved in the clearing of Fulton St. and the creation of the plaza are of considerable importance to Brooklyn in improving a region which is now the focal point of the city and beautifying it very considerably. The reconstruction of the bridge station and trackage, however, is fully as important, and in connection with the improvement of the Manhattan terminal (previously described) and the operation of bridge trains through the Loop Subway it is important in modernizing a transportation system which has been of fundamental importance to New York City for a quarter century.

A Submarine Telephone Cable equipped with Pupin coils has been laid by the British Post Office between England and Belgium. The conductors are of the same size as those of a similar cable laid between England and France last year (160 lb. per nautical mile). The insulation has been reduced, however, from 300 lb. per mile to 150, a special gutta-percha being used. A further advance in cable construction has been made by arranging the two circuits in the cable so that they may be used for three simultaneous conversations. This is now termed "phantoming" (see "Engineering News," May 25, 1911). The actual construction of the cable follows the method employed in the French cable noted. The loading coils were inserted one mile apart and the additional loading coil for the phantom circuit was also inserted there. This additional coil is similar to the others except that one of the windings is reversed; it increases the length of a bulge in the cable by some 8 inches. The cable was landed at St. Margaret's, England, and Lapanne, Belgium covering a distance of 47.9 miles.

Largest Dredging Plant in the World

What is without doubt the largest dredging fleet in the world is that owned and operated by the United States Government and engaged on river and harbor improvement work under direction of the Corps of Engineers. This fleet comprises 137 dredges and 855 vessels of all classes. During the year ending June 30, last, the seagoing hopper dredges and hydraulic pipe line dredges of this fleet excavated over 52 million cubic yards of material or about five-sevenths as great a volume as the entire contents of the Culebra cut, on the Panama Canal. Some of this dredging last year was done at a total cost as low as 1¾c. per cu.yd. The average cost for all the material handled by these dredges was only 5.40c. per cu.yd. for the hopper dredges and 7.18c. for the pipe line dredges.

There are probably few, even among engineers, who realize what a huge fleet of vessels is now owned and operated by the United States government upon the works of river and harbor improvement. These works are all in charge of the Corps of Engineers of the Army, but the floating equipment employed upon them is nearly as numerous as the vessels of the navy. The complete list of vessels, as given in the annual report of Gen. Wm. H. Bixby, Chief of Engineers, U. S. A., is as follows:

TABLE I

	No.
Sea-going hopper dredges	23
Hydraulic pipe-line dredges	53
Bucket and dipper dredges	61
Snag boats	60
Screw tug and survey boats	66
Paddle tow and survey boats	98
Steam lighters	5
Launches	204
Maneuver boats	17
Derrick boats	54
Pile drivers	49
Graders	7
Drill boats	9
Quarter boats	146
Floating drydocks	3
Total vessels	855

The annual report of the Chief of Engineers contains a great deal of matter of interest and usefulness to the profession, but it is so hidden in the bulky volumes and by the formal style in which the reports are presented that very few engineers, even among the officers of the corps, ever have the time and patience to dig it out.

A valued correspondent of ENGINEERING NEWS, however, has had the patience to search through the report for the year ending June 30, 1911, and glean from it the matter relating to the year's work by the government's fleet of hopper seagoing and pipe-line dredges, and has summarized it in the accompanying table.

He also presents comments on the dredging work at various points as follows:

Of the seagoing hopper dredges, 14 were engaged at harbors along the Atlantic coast, 7 along the coast of the Gulf of Mexico, 2 along the Pacific coast, 2 in the Delaware river, and 1 each in Lakes Erie and Michigan. The total amount of material dredged by the 23 seagoing hopper dredges, during the year 1910-11, was 31,803,546 cu.yd., at a total cost of $1,719,314, or an average cost of 5.4c. per cu.yd.

The cost at the various points ranges from 1.73c. as a minimum, to 13.4c. as a maximum, if we except the dredge "Charleston," which only worked 14 days at Mobile, Ala., and was then transferred to Charleston, S. C., and the dredge "Manhattan," which only worked 16 days at New York, and was then transferred to Philadelphia.

WORK IN AMBROSE CHANNEL, NEW YORK HARBOR

At the beginning of the fiscal year July 1, 1910, the four U. S. dredges, "Manhattan," "Atlantic," "Raritan" and "Navesink," were at work upon the Ambrose Channel excavation.

As this channel was nearing completion, two of the dredges were leased to other districts at a rental of $3000 per month. The dredge "Manhattan" was sent to the Delaware River and the dredge "Atlantic" to the South West Pass, Mississippi River. The four dredges dredged in the Ambrose Channel to a depth of from 35 to 42 ft., excavating 5,710,739 cu.yd. of material, consisting of 95% sand and 5% mud and stone. The material was carried to sea and dumped in deep water, at a distance from the dredging site averaging about 10½ miles.

Each dredge has two hoppers. The total capacity of those of the "Atlantic" and "Manhattan" is 2476 cu.yd. each, and of the "Navesink" and "Raritan," 2884 cu.yd. each.

TABLE II
DETAILS OF OPERATIONS

Dredge	Loads	Cu.yd.
Atlantic	309	657,129
Manhattan	50	106,067
Navesink	983	2,576,237
Raritan	963	2,499,231
Totals	2,305	5,838,664

The total cost of the work was $345,762, including all office expenses, equal to 6.05c. per cu.yd. dredged.

The working time of the dredges "Navesink" and "Raritan" was 11,659 hr. This includes pumping, turning in cut, going to and from dump and dumping, and going to and from wharf. It does not include taking on fuel and other supplies, repairs, storms, fog and similar delays. Excluding the 59 Sundays and holidays, the average actual working time in the channels made by each dredge was 19.05 hr. per day. The average amount dredged per day for the "Navesink" was 8822 cu.yd. (292 days) and for the "Raritan," 8387 cu.yd. (298 days).

ANALYSIS OF LOW-COST RECORDS FOR DREDGING

The cheapest dredging was done in the Delaware River at various localities between Philadelphia Harbor and Delaware Bay, the seagoing hopper dredges "Delaware" and "Manhattan" being used in the work. The greater part of the shoaling which takes place in the channel of the Delaware River is below Schooner Ledge. It is composed of very soft mud, and it is difficult to fill the dredges' bins with solid material when working in such localities. The dredges when operating in this locality have pumped directly overboard on the ebb tide, so that the light material would be carried by the tide toward deep water in Delaware Bay. On flood tide the material was held in the dredge bins and deposited at Deep Water Point, N. J., and back of Artificial Island, to be rehandled.

The total amount of material removed during the year and the cost of the work are as follows:

TABLE III

	"Delaware," cu.yd.	"Manhattan," cu.yd.
Dredged and deposited	1,129,678	1,215,672
Dredged and pumped overboard	3,986,647	3,168,053
Total	5,116,325	4,383,725
Cost of operation	$96,964	$74,916
Office expenses	1,200	1,100
Total	$98,164	$76,016
Total cost per cu.yd.	1.92c.	1.73c.

The dredge "Delaware" worked on 284 days during the year, the average output per day being 18,015 cu.yd. The working time was equivalent to 249 days of 24 hr. each, the lost time 57 days, a total of 306 days, the remaining 59 days being Sundays and holidays.

The dredge "Manhattan" worked on 242 days, the average output per day being 18,114 cu.yd. The working time was equivalent to 214 days of 24 hr. each, the lost time 59 days, a total of 273 days, the remaining 92 days being at New York, Sundays and holidays.

The various items making up the cost of operations for the dredge "Delaware" are as shown in Table IV:

TABLE IV
DISTRIBUTION OF COST AND PERCENTAGE OF TOTAL COST

	Amount	Per cent.
Pay roll	$36,244	36.90
Coal	33,336	34.00
Supplies:		
Subsistence	9,193	9.40
Engine room	2,450	2.50
Miscellaneous	1,624	1.70
Renewals of or additions to outfit	610	0.60
Ordinary repairs	3,137	3.20
Extraordinary repairs	9,340	9.50
Alterations and additions to vessels	792	0.80
Laundry and miscellaneous expenses, wharfage, etc.	238	0.20
Office expenses	1,200	1.20
Total cost	$98,164	100.00
Average cost per cubic yard		1.92c.

TABLE V

OPERATIONS OF DREDGING PLANTS OWNED AND OPERATED BY THE ENGINEER DEPARTMENT AT LARGE, U. S. ARMY, FOR THE FISCAL YEAR ENDING JUNE 30, 1911.

CLASS A.—SEAGOING HOPPER DREDGES.

Dredge	When Built, Year	Contract Cost and Outfit	Dimensions of Hull Length Beam Depth	Dredging Pumps Inches	Pumping Engines Compound No. Inches	Discharge Pipe Diam. Inches	Number and Type Boilers	Crew Men	Location of Dredging	No. of Days Dredging Was Done	Distribution of Time Worked Hrs.	Distribution of Time Lost Hrs.	Character of Dredged Material	Material Dredged During Year Cubic Yards	Total Cost of Dredging	Total Cost Per Cubic Yard Cents
Atlantic	1905	$399,077	288 x 48 x 25	2-20	2-16-32 x 18		4 Scotch	57	New York	89	1,999	689	Sand, Mud, Stone	657,129	$55,837	8.34
Atlantic	1905	.....	.....	.....	.....		.....	..	S. W. Pass-Miss. R.	183	3,481	910	Sand, Silt, Mud	1,043,141	62,190	5.96
Benyaurd	1904	414,238	272 x 48 x 23	2-20	2-16-32 x 20		4 Scotch	63	S.W. Pass-Miss.R.	257	5,447	1,897	Sand, Mud, Silt	1,983,494	129,010	6.50
Burton	1905	184,300	177 x 38 x 19	2-15	2-12-22 x 14		2 Scotch	28	Lake Erie	143	1,394	.....	Sand, Mud, Clay	201,662	27,214	13.49
Cape Fear	1895	116,589	131 x 29 x 12	2-10	2-10½ x 10½		1 Scotch	16	Cape Fear R.	124	2,487	789	Sand, Shell, Mud	373,500	22,157	5.93
Cape Fear	1895	.....	.....	.....	.....		.....	..	Beaufort, N. C.	114	1,327	.....	Coarse Sand	237,003	14,987	6.32
Caucus	1905	176,000	200 x 41 x 23	2-18	2-14-26 x 18		2 Scotch	34	Pensacola, Fla.	224	2,482	4,862	Sand	1,001,209	49,260	4.92
Charleston	1891	69,000	123 x 30 x 12	1-15	1-13-23 x 14		1 Scotch	25	Charleston, S. C.	155	1,828	1,013	Sand, Shell, Mud	347,027	38,946	11.22
Charleston	1891	.....	.....	.....	.....		.....	..	Mobile, Ala.	14	196	548	Fine Sand	32,184	11,679	36.29
Chinook*	1910	354,799	460 x 49 x 34	2-20	*2-13-20-31-20		4 Scotch	45	Columbia R.	160	1,697	889	Fine Gray Sand	722,431	70,314	9.74
Clatsop	1908	234,500	180 x 38 x 23	2-18	2-12-26 x 18		2 Scotch	44	Columbia R.	231	4,343	3,001	Sand	2,352,787	90,305	3.84
Comstock	1895	96,000	177 x 37 x 16	2-15	2-12-22 x 14		2 Scotch	41	Cape Fear R.	235	3,815	3,529	Sand, Clay, Shells	1,508,414	83,811	5.60
Cumberland	1902	181,874	200 x 41 x 21	2-18	2-14-26 x 16		2 Scotch	35	Savannah, Ga.	167	1,922	5,422	Sand, Mud, Gravel	667,580	78,242	11.73
Delaware	1905	392,134	315 x 52 x 23	2-20	2-16-32 x 18		4 Scotch	59	Delaware R.	284	5,976	1,368	Sand, Clay, Gravel	5,116,325	98,164	1.92
Galveston	1908	380,435	304 x 51 x 27	2-20	2-20-17-36-18		4 Scotch	34	Galveston, Tex.	214	2,771	4,033	Sand	1,759,789	125,881	7.15
Gedney	1887	72,500	157 x 37 x 16	2-15	2-15-20		2 Scotch	21	Fall River, Mass.	188	2,392	4,952	Mud, Sand, Cobble	273,138	34,772	12.73
Key West	1904	101,708	142 x 32 x 15	1-15	1-12-22 x 14		1 Scotch	22	Key West-Tampa	236	2,376	4,968	Coral Sand, Mud	457,501	35,497	7.76
Manhattan	1904	399,077	288 x 48 x 25	2-20	2-16-32 x 18		4 Scotch	57	New York	16	326	490	Sand, Mud, Stones	106,067	30,548	28.80
Manhattan	1904	.....	.....	.....	.....		.....	..	Delaware R.	242	5,134	1,418	Mud, Sand, Gravel	4,383,725	76,016	1.73
Meade, Gen. G. G	1904	177,953	177 x 38 x 19	2-15	2-12-22 x 14		2 Scotch	31	Lake Michigan	194	2,090	1,056	Sand, Gravel, Stone	466,536	52,003	10.48
Navesink	1908	406,044	290 x 48 x 28	2-20	2-17-36 x 18		4 Scotch	59	New York	292	6,377	967	Sand, Mud, Stone	2,576,237	124,691	4.84
Raritan	1908	410,304	290 x 48 x 28	2-20	2-17-36 x 18		4 Scotch	59	New York	298	6,489	855	Sand, Mud, Stone	2,499,231	116,927	4.68
Sabine	1901	84,833	145 x 35 x 12	2-10	2-10 x 11		2 Scotch	17	Sabine Pass, Tex.	194	1,907	5,437	Soft Mud, Silt	507,488	35,666	7.03
Savannah	1904	195,085	177 x 38 x 19	2-15	2-12-22 x 14		2 Scotch	34	Savannah, Ga.	218	2,477	4,867	Sand, Clay, Mud	721,193	60,085	8.33
St. Johns	1905	176,000	207 x 41 x 20	2-18	2-14-26 x 18		2 Scotch	39	St. Johns R., Fla.	187	2,051	5,293	Sand, Shell	821,321	93,203	11.35
Sumter	1904	151,834	200 x 41 x 22	2-18	2-14-26 x 18		2 Scotch	40	Charleston, S. C.	166	1,977	5,367	Sand, Mud, Shell	673,782	67,418	10.00
Winyah Bay	1898	86,000	141 x 32 x 14	1-15	1-12-22 x 14		1 Scotch	24	Charleston, S. C.	182	2,002	5,342	Fine Hard Sand	313,651	34,482	10.99
Totals		$5,260,284			* Triple Expansion.									31,803,546	1,719,314	5.40

CLASS B.—HYDRAULIC PIPE LINE DREDGES.

Dredge	When Built, Year	Contract Cost and Outfit	Dimensions of Hull Length Beam Depth	Fuel	Pumping Engines Compound No. Inches	Discharge Pipe Diam. Inches	Number and Type Boilers	Crew Men	Location of Dredging	No. of Days Dredging Was Done †	Distribution of Time Worked Hrs.	Distribution of Time Lost Hrs.	Character of Dredged Material	Material Dredged During Year Cubic Yards	Total Cost of Dredging	Total Cost Per Cubic Yard Cents
Barnard	1904	$230,000	206 x 38 x 14	Coal	*1-21-32-54-26	36	2 Scotch	41	Gulf Port, Miss.	109	1,233	795	Mud	1,156,938	54,927	4.75
Cataract	1905	94,811	141 x 40 x 11	"	*1-12-22-33-20	20	3 Scotch	35	Delaware R.	...	4,491	3,433	Sand, Clay, Gravel	1,398,069	45,027	3.22
Congaree	1908	31,685	111 x 33 x 6	Wood	1-10-20 x 10	12	2 Locomotive	12	Congaree R., S.C.	...	.....	.....	Sand	100,000	11,107	11.11
Delatour	1909	40,812	112 x 30 x 5	Oil	1-12-22 x 14	13	2 Flue	15	Louisiana	...	1,761	1,743	Mud, Silt, Shells	163,118	23,219	14.23
Epsilon[1]	1898	105,000	157 x 40 x 8	Coal	2-16-26 x 18	12	6 Miss. R.	35	Mississippi R.	130	399	73	Sand, Gravel, Silt	271,076	31,398	9.90
Etna	1909	29,733	130 x 28 x 5	"	1-12-23 x 16	12	3 Flue	16	Mississippi R.	...	2,298	2,015	Sand, Mud, Gravel	315,750	22,482	7.10
Flad, Henry[1]	1901	135,400	192 x 44 x 7	"	2-15-30 x 24	32	7 Miss. R.	43	Mississippi R.	147	419	92	Sand, Gravel, Drift	230,910	27,516	11.92
Florida	1905	90,000	152 x 30 x 7	"	1-12-20 x 12	12	2 Clyde	21	St. Johns R., Fla.	...	2,929	1,559	Sand, Mud, Clay	633,131	52,126	8.23
Ft. Chartres[2]	1908	176,796	197 x 45 x 7	"	2-16-30 x 24	32	6 Miss. R.	52	Mississippi R.	166	805	3,179	Sand, Gravel	960,250	69,242	7.21
Fort Gage[2]	1908	177,979	197 x 45 x 8	"	2-16-30 x 24	32	6 Miss. R.	52	Mississippi R.	163	704	3,206	Sand, Gravel	801,350	58,588	7.31
Gamma[1]	1897	88,531	138 x 38 x 8	"	1-18-32 x 22	24	6 Miss. R.	36	Mississippi R.	113	469	488	Sand, Mud, Gravel	264,593	40,635	15.45
Geyser	1893	4,704	100 x 20 x 4	"	1-13 x 14	12	1 Flue	15	Mississippi R.	...	2,967	828	Sand, Clay, Gravel	296,095	12,833	4.30
Harrod, B. M.[1]	1907	241,998	210 x 44 x 9	"	2-20-34 x 24	36	9 Miss. R.	47	Mississippi R.	153	585	135	Sand, Gravel, Mud	841,133	27,516	3.28
Hecla	1901	27,084	120 x 26 x 5	"	2-14-24 x 15	18	2 Tubular	15	Mississippi R.	...	2,261	1,578	Sand, Mud, Gravel	234,860	17,484	7.44
Howell, Capt. C. W	1904	40,000	125 x 26 x 5	Oil	1-12-22 x 14	12	2 Scotch	35	Galveston, Tex.	...	4,795	3,965	Sand, Shell, Clay	524,525	57,638	10.44
Humphreys, Chas	1908	46,561	100 x 32 x 9	Coal	1-12-22 x 14	12	2 Steamboat	20	Black Warrior R.	...	1,788	2,093	Sand, Gravel	122,576	14,934	12.18
Indiana	1908	116,638	125 x 34 x 7	"	1-12-22 x 14	15	1 Worthington	28	Lower Ohio R.	...	1,040	983	Sand, Gravel, Boulders	264,612	26,763	10.10
Iota[1]	1900	103,480	192 x 44 x 7	"	2-16-26 x 20	33	7 Miss. R.	43	Mississippi R.	143	455	155	Sand, Gravel, Lignite	238,768	28,238	11.83
Jacksonville	1904	125,000	138 x 40 x 10	"	*1-17-28-42 x 20	32	4 Babcock	30	Cape Fear R.	...	3,177	1,903	Sand, Mud, Clay	1,119,679	79,734	8.70
Kappa[1]	1901	135,400	192 x 44 x 7	"	2-15-30 x 24	32	7 Miss. R.	43	Mississippi R.	144	969	313	Sand, Mud, Drift	519,019	29,415	5.67
Miller, Col. A. M	1906	112,952	139 x 37 x 13	Oil	*1-14-23-36 x 18	20	2 Heine	47	Galveston, Tex.	305	4,768	3,992	Mud, Sand, Clay	1,333,927	116,686	8.75
Morgan	1908	151,176	134 x 38 x 8	Coal	*1-12-20-32 x 20	20	2 Scotch	43	Savannah, Ga.	...	4,616	3,601	Mud, Clay, Shell	1,113,535	82,742	7.43
Muscogee	1904	12,900	120 x 30 x 8	"	1-12-20 x 12	12	1 Scotch	10	Chattahoochee R.	...	1,504	773	Sand, Gravel, Marl	104,440	19,521	18.70
Oregon	1905	160,614	120 x 36 x 11	Wood	1-18-28 x 18	18	2 Heine	24	Coos Bay, Ore.	...	1,447	882	Sand, Shell, Clay	446,654	33,064	7.40
Oregon	1908	.....	.....	"	.....	..	.....	..	Grays Harbor	...	519	457	Mud, Clay, Gravel	197,852	12,757	6.45
Pascagoula	1909	196,832	150 x 40 x 11	Coal	*1-12-20-33 x 20	20	2 Scotch	41	Pascagoula R.	...	4,778	3,982	Mud, Sand, Shells	1,719,013	97,409	5.67
Pelee	1909	36,172	119 x 30 x 5	"	1-12-24 x 14	17	2 Hawkes	18	Mississippi R.	...	.....	.....	Sand, Gravel	448,995	19,075	4.30
Pettus	1909	62,352	135 x 35 x 7	"	1-13-20 x 16	15	3 Western R.	34	Alabama R.	...	1,874	2,526	Sand, Gravel	199,177	21,776	10.93
Port Arthur	1901	12,000	60 x 28 x 6	Oil	2-10-16 x 8	12	2 Scotch	15	Port Arthur, Tex.	...	5,638	1,748	Silt, Clay, Sand	247,084	24,140	9.76
Ram	1893	104,150	125 x 30 x 7	"	1-14-28 x 20	16	2 Gunboat	16	Red River, La.	...	.....	.....	Sand, Gravel	85,000	10,595	12.46
San Pedro	1905	118,722	141 x 41 x 11	"	*1-12-20-33 x 20	20	2 Babcock	36	Los Angeles, Cal.	...	5,305	3,114	Sand, Rock, Clay, Silt	764,250	69,673	9.10
Selma[2]	1898	117,151	160 x 40 x 6	Coal	*2-15-26 x 18	28	6 Miss. R.	54	Mississippi R.	190	922	3,470	Sand, Gravel	856,700	55,155	6.44
St. Croix	1898	8,621	80 x 22 x 5	Wood	1-10 x 12	10	1 Scotch	11	Minnesota R.	...	1,085	585	Gravel, Sand	8,678	5,072	58.45
Thebes[2]	1898	114,575	160 x 40 x 6	Coal	2-15-26 x 18	28	6 Miss. R.	54	Mississippi R.	239	1,289	1,279	Sand, Gravel, Clay	546,000	51,852	9.50
Tortoise	1905	27,170	102 x 24 x 5	"	2-12 x 12	15	2 Locomotive	7	Superior, Wis.	...	2,360	1,069	Sand, Clay, Peat	189,666	29,857	15.74
Uncle Sam	1899	22,064	85 x 34 x 7	"	1-12-26 x 12	12	1 Scotch	14	Delaware R.	...	1,573	2,807	Mud, Sand, Cobble	238,259	14,324	6.00
Vesuvius	1909	33,032	115 x 30 x 5	"	1-12-24 x 14	18	2 Bonson	16	Mississippi R.	...	2,394	1,061	Sand, Mud, Gravel	224,604	15,598	6.95
Wahalak	1910	186,027	150 x 40 x 12	"	*1-14-23-40 x 20	20	4 Scotch	44	Mobile Bay	99	1,493	877	Mud, Sand	1,005,935	24,700	2.46
Warroad	1904	33,862	100 x 27 x 9	Wood	2-9 x 9	12	2 Scotch	11	Warroad R.	...	1,125	395	Sand, Clay, Muskeg	86,144	7,104	8.25
Zeta[1]	1898	112,000	157 x 40 x 8	Coal	2-16-26 x 18	32	6 Miss. R.	35	Mississippi R.	57	233	64	Sand, Mud, Gravel	105,541	21,184	20.00
		3,663,984			* Triple Expansion.					† Days in Commission.				20,378,006	1,463,106	7.18

REMARKS:—In addition the Department owns the following hydraulic pipe-line dredges, which were not in commission during the year, viz., Augusta, Beta, Blackwater, Dalecarlia, Delta, Deluge, Gen. Ludlow, Macon, No. 1—O.R., No. 3, No. 569, Pump Boat No. 1, Suwanee and Noname.

NOTE[1]—Dredges Epsilon, Henry Flad, Gamma, B. M. Harrod, Iota, Kappa and Zeta, under Mississippi River Commission below mouth of Ohio River, at Cairo, Ill.

NOTE[2]—Dredges Fort Chartres, Fort Gage, Selma and Thebes, Mississippi River, between mouths of Ohio and Missouri Rivers.

None of the other seagoing hopper dredges show such low records of total cost per cubic yard as the "Manhattan" and "Delaware," the large amount of material dredged and pumped overboard (not transported) accounting for the small unit cost.

The dredges having a record approximating a cost of 5c. per cu.yd., are the "Clatsop," 3.84; "Raritan," 4.68; "Navesink," 4.84; "Caucus," 4.92; "Comstock," 5.60, and "Atlantic," 5.96c. Of the two dredges employed on the Pacific Coast, the "Clatsop" shows the lowest unit cost.

This seagoing hopper dredge, which was built at Newport News, Va., started from Philadelphia, in Dec. 9, 1908, for a 17,000-mile trip around Cape Horn to Portland, Orc. It reached that port on Apr. 20, 1909, under its own steam, in 132 days, 54 of which were spent at various ports taking on coal and supplies. Its average steaming rate per day was 218 miles, or at the rate of 9 miles per hour.

This dredge worked throughout the year—1910-1911—at various bars between Mount Coffin and Astoria in the improvement of the Columbia River, removing a total of 2,353,626 cu.yd. of material from the channel and depositing it in deep holes outside the channel. The dredge worked on 231 days, out of 306, the equivalent working time being 181 days, the lost time 125 days, not including 59 Sundays and holidays. It is intended to change this dredge from a coal to a fuel-oil burner, plans and specifications having already been prepared for the necessary additions and alterations.

There are two seagoing hopper dredges employed on the Great Lakes, the "Burton," on Lake Erie, and the "Gen. G. G. Meade," on Lake Michigan. The "Gen. Meade" was built in 1904, at Sparrows Point, Md., by the Maryland Steel Co. She has two hoppers with a total capacity of 935 cu.yd. Since Mar. 1, 1909, the work of this dredge has been closely supervised and verified. Under extremely favorable conditions a maximum of slightly more than 4000 cu.yd. has been removed in a day of eight hours' actual dredging, but the average at all harbors and under all circumstances is about 2600 cu.yd. per day. For dredging under precisely the same conditions in the past the contract cost with dipper dredges has been as high as 20c. per cu.yd., and then work outside the piers was never possible until about May 1, whereas the "Meade" works in the most exposed places at practically any and all seasons. During the fiscal year the "Meade" was in operation July 1, 1910, to Jan. 16, 1911, and Mar. 21 to June 30, 1911, except while an oil-burning apparatus was being installed and machinery and hull repaired. Oil fuel has been in use since June 13, 1911. The main reasons for making the change from coal to oil were to reduce the great cost due to lost time coaling and to reduce the fireroom force. It was not expected to reduce the cost of fuel.

The final average cost of the work of the dredge for the fiscal year—1911—is 11.19c. per cu.yd. and this is believed to fairly represent the cost of dredging under the conditions existing in the Grand Rapids, Mich., district.

When compared with the performances of similar dredges on the sea coast, the cost may seem high, but the circumstances under which this dredge is obliged to work affect unfavorably the final cost per unit. When loaded, the dredge draws too much for many of the lake harbors (Saugatuck, Ludington, Frankfort, Charlevoix, Mich., and others). The dredge cannot, therefore, steam freely over the shoals, and is often obliged to proceed to the dumping grounds with bins only partly loaded. Moreover, the channels are so narrow that the dredge often has to back to the site of the work. Quantities to be dredged at each harbor are generally small and to the cost of actual dredging must be added that of moving over a district the extreme harbors of which are 280 miles apart.

Contract vs. Government Dredging

Referring to Engineering News, Jan. 11, 1912, Construction News Supplement, p. 29, we note the following bids received by Capt. T. H. Jackson, Corps of Engineers, U. S. Army, for 7,291,000 cu.yd. of dredging in the Sabine-Neches Canal, Tex.:

TABLE VI

	Cents per cu.yd.
Southern Dredging Co., Mobile, Ala.	10.40
Atlantic, Gulf & Pacific Co., New York	8.25
North American Dredging Co. of Texas, Galveston, Tex.	8.14
Duluth-Superior Dredging Co., Duluth, Minn.	7.75
Bowers Southern Dredging Co., Galveston, Tex.	7.29

The following are the operations of the seagoing hopper dredge "Sabine," working in the Sabine Pass Jetty channel, Tex. and La., 1911:

TABLE VII

	Amount	Per cent.
Pay roll	$10,949	30.698
Fuel oil	5,354	15.012
Water	500	1.402
Supplies:		
Subsistence	2,181	6.104
Engine room	874	2.449
Renewals of or additions to outfit	398	1.117
Ordinary repairs	268	0.751
Extraordinary repairs	13,200	37.016
Laundry and miscellaneous expenses, wharfage, etc.	464	1.307
Office expenses	1,478	4.144
Total cost	$35,666	100.000

The amount of material dredged (soft blue mud and silt) was 507,488 cu.yd., costing 7.03c. per cu.yd.

Of the 23 seagoing hopper dredges, 5 are oil and 18 coal burners. The price of the oil burned ranged from 75c. to $1.31 per bbl., and the fuel consumed per cubic yard of dredged material from 4 to 7 lb. for the oil burners. The dredge "Galveston" consumed the largest amount of fuel oil, namely, 39,268 bbl., costing $1.0739 per bbl.

The cost of the coal burned ranged from $3.04 to $6.67 per ton and the fuel consumed per cubic yard of dredged material from 4.27 to 15.7 lb. for the coal burners. That of the "Manhattan" was 4.27, of the "Navesink" 9.94, of the "Raritan" 9.73 lb.

The average time to dredge one load ranged from 32 min. for the "Sabine," to 3 hr. 22 min. for the "Raritan," the hopper capacity being 325 and 2884 cu.yd., respectively, and the average time spent going to and from dump per load, from 13 min. for the "Charleston," to 2 hr. 16 min. for the "Raritan," the distances from work to the dump being ½ and 12½ miles, respectively.

The dredge "Atlantic," working in the Southwest Pass, Mississippi River, burned 57,959 "barrels" of coal and the "Benyaurd" 99,411 "barrels." This is a local measure peculiar to the Mississippi River, the contract price ranging from 35c. to 39c. per barrel, depending upon the point of delivery.

The cost of building the 23 seagoing hopper dredges was $5,260,284; of these, seven cost in the neighborhood of $400,000 each.

Hydraulic Pipe-line Dredges

Table V, on page 887, also shows the operations of 40 hydraulic pipe-line dredges, one—the "Oregon"—showing its employment in two engineer districts. The amount of material dredged for the year 1910-11 was 20,378,006 cu.yd., costing $1,464,106, or an average cost of 7.18c. per cu.yd. Seven of these dredges show an output of over 1,000,000 cu.yd. each; for the 40 dredges the average output was a little over 500,000 cu.yd.

On June 20, 1896, the Mississippi River Commission adopted a project for obtaining and maintaining, by means of dredges, a channel in the Mississippi River below Cairo, with a width of 250 ft. and a depth of at least 9 ft. throughout the year, except when the river is closed by ice. The Commission in 1891 took under consideration the question of construction of dredging devices to be operated with a view to obtaining temporary relief for low-water navigation and experimented along these lines between 1891 and 1896.

Up to the present time 10 hydraulic dredges have been constructed, 9 of which are still serviceable. The floating plant for this work comprises beside the 9 dredges, 10 steamboats, 5 pile sinkers, 2 barges, 1 pump boat, 2 calking flats and 6 sectional dock sections. Also a number of skiffs and small motor boats. The rendezvous of the dredge fleet is the depot at West Memphis, Ark.

During the low-water season of 1910, seven dredges, the "Gamma," "Epsilon," "Zeta," "Iota," "Kappa," "Henry Flad"

TABLE VIII

TABLE SHOWING COST OF DREDGING IN MISSISSIPPI RIVER, BETWEEN MOUTHS OF OHIO AND MISSOURI RIVERS

Items	Pipe-line hydraulic dredges "Selma"	"Thebes"	"Ft. Gage"	"Ft. Chartres"
Pay rolls	$13,971.01	$11,985.33	$12,505.60	$12,452.07
Coal for boilers	6,050.69	4,173.21	7,235.55	8,476.07
Coal for galley	93.97	122.73	83.70	85.75
Supplies, subsistence	4,401.35	3,989.12	3,989.20	4,054.31
Supplies, engine room	1,302.00	1,821.53	760.92	931.23
Renewals and additions to outfit	4,538.33	.....	.....	.....
Ordinary repairs	204.85	4,902.61	4,125.15	10,657.37
Laundry, ice, miscellaneous expenses	.....	259.21	208.09	223.02
Towboats and launches	3,278.44	4,281.68	2,920.14	2,991.80
Repairs, barges	439.20	573.60	391.20	400.80
Office expenses, superintendence, surveys	1,904.86	1,904.86	1,904.86	1,904.86
Extraordinary repairs, dredges	67.48	301.53	.....	.....
Extraordinary repairs, towboats and launches	404.00	404.00	.....	.....
General expense	13,873.72	12,882.69	14,534.36	17,176.98
Deterioration	4,625.45	4,250.38	9,929.72	9,888.38
Grand total	$55,155.35	$51,852.48	$58,588.49	$69,242.64
Cubic yards dredged	856,700	546,100	801,350	960,250
Gross cost per cu.yd	6.44c.	9.50c.	7.31c.	7.21c.

and "B. M. Harrod," were operated at various crossings from Medleys, 31 miles below Cairo, to Youngs Point, just above Vicksburg, or 597 miles below Cairo. The dredging operations thus extended over a distance of 566 miles measured on the line midway between banks. In this stretch of river the dredging operations were distributed among 20 different shoals. In addition to the shoals being numerous and widely scattered, many of them were exceedingly troublesome and required repeated dredging. The dredging operations were carried on between Aug. 3, 1910, and Jan. 2, 1911, the season of low water in the Mississippi River.

Table IX shows in the first column the name of the dredge, the second column the number of days laid up at the dredge depot at West Memphis, Ark., the third column the working days (of 24 hr.) while engaged in dredging, the fourth column the idle time, days of 24 hr., while dredging.

TABLE IX

DISTRIBUTION OF TOTAL TIME OF DREDGES FOR THE YEAR 1910

Name of dredge	Days laid up	Dredging Days working	Dredging Days idle
"Gamma"	252	47	66
"Epsilon"	235	48	82
"Zeta"	308	27	30
"Iota"	222	29	114
"Kappa"	221	51	93
"Henry Flad"	218	27	120
"B. M. Harrod"	212	38	115

The actual gross cost of dredging per cubic yard for the above dredges, given in the order as stated, was: 15.45, 9.90, 20.0, 11.83, 5.67, 11.92 and 3.27c.

Taking as a basis the cost of actual time dredging, placing plant, changing cuts, passing boats, and making up tow, the cost per cubic yard for the different dredges would range from 1 to 3.2c. per cu.yd.

In addition to the seven hydraulic pipe-line dredges working in the Mississippi River below the mouth of the Ohio River (Cairo), four dredges were in commission in the Mississippi River, between the mouths of the Ohio and Missouri Rivers, throughout the entire low-water season of navigation, July 1 to Dec. 13, 1910, and operated upon 25 bars which developed in that time.

The dredges were in commission and service as follows; season 1910-1911:

TABLE X

Dredge	Dates	Days In commission	Days Actual dredging
Selma	July 7-Dec. 6, May 25-June 30.	190	74
Thebes	July 26-Dec. 10, Mar. 22-June 30.	239	103
Ft. Gage	July 1-Dec. 10..	163	78
Ft. Chartres	July 1-Dec. 13...	166	91

The total number of channels dredged at the 23 bars was 76, having a combined length of about 19 miles, and an average width of 200 ft. The total amount of material dredged was nearly 3,500,000 cu.yd. in 3805 dredge hours, an average of 900 cu.yd. per hour. The average rate of sand excavation per hour for the 28-in. pumps of the dredges "Selma" and "Thebes" was nearly 800 cu.yd., and for the 32-in. pumps of dredges "Ft. Gage" and "Ft. Chartres," nearly 1100 cu.yd. The amounts of excavation were measured in place, from soundings before and after the passage of the dredge.

As already stated, the dredges were engaged in actual excavation about one-fifth of their time in commission. The final cost of dredging was about 7c. per cu.yd. of sand moved, including the cost of idle time, upkeep, and repairs during the year. If the dredges had been run for the entire time in commission, the expense would have been augmented only by the increased amount of fuel, oil, etc., consumed, and the cost for such continuous operation with the same average output would have been slightly over 1c. per cu.yd.

As the dredges must be held in readiness for work as it offers, depending upon fluctuations, stages, and changes in river conditions, wide variation in cost of work from year to year cannot be avoided. During the total dredging period, 177 days, there were 8 days when all four dredges were working, 45 days when three were working, 36 days when two were working, 36 days when one was working, and 52 days of no dredging.

In the cost of the work are included all expenses of dredge, attending tow boats, launches and barges, office expenses, superintendence, surveys, ordinary and extraordinary repairs to plant, fuel and supplies. With these items the average cost per cubic yard is 4.67c.

Including the proportionate charges of engineer office, general expense and care and use of plant, engineer depot, tow boats, and deterioration which is properly chargeable to operation and care of the dredge, the gross cost of output per cu.yd. (average for four dredges) is 7.42c.

The length of the discharge pipes of the dredges "Selma" and "Thebes" is 300 ft., that of the dredges "Ft. Gage" and "Ft. Chartres," 440 ft. Each dredge uses nine pontoons and two coal barges for fuel. The dredged material is sand and gravel.

In addition to the hydraulic pipe-line dredges enumerated, there are five others employed in dredging in the Mississippi River between the mouth of the Missouri River and Minneapolis, Minn., namely, the "Etna," "Geyser," "Hecla," "Pelee" and "Vesuvius." The amount of material excavated by these dredges was 1,520,304 cu.yd., sand, mud, clay and gravel, costing $87,473, or 5¾c. per cu.yd.

Of the 40 dredges whose operations are given in the table, 16 are employed on the Mississippi River; the other 24 were employed in the Delaware River, Atlantic, Gulf and Pacific coasts, Ohio and other interior rivers.

The cost of building the 39 dredges (the "Oregon" worked at two localities) was $3,663,974. The "B. M. Harrod" was the most expensive, namely, $242,000. This dredge and the "Barnard" have discharge pipes of 36 in. diameter.

The cheapest dredging was done by the dredge "Wahalak," operating in Mobile Harbor, from Mar. 24 to June 30, 1911, in all 99 days. During this period the dredge was engaged in dredging the cut-off channel at Beacon 16 in Mobile Bay, the length of the cut formed was 7341 ft., the width varied between 130 and 160 ft., and the depth 27 to 32 ft., the amount of material excavated was 1,005,935 cu.yd. of mud and sand, or an average of 10,161 cu.yd. per 24-hr. day. The total pumping time was 1316 hr., at the rate of 764.4 cu.yd. per hour. The material was measured in place, and the gross cost was 2.46c. per cu.yd. The average depth before dredging was 5.5 ft., and after dredging 28 ft., the rate of advance per hour was 5.6 ft.

The total cost of this dredging in Mobile Bay is apportioned as follows:

TABLE XI

Dredge	$20,468.90
Towboats and launches	2,965.91
Office expenses, superintendence, surveys, etc	1,239.44
Extraordinary repairs, barges	26.40
Total	$24,700.65

The First Reinforced-Concrete Arch Bridge in Japan

The first reinforced-concrete bridge in Japan is the new Yoshida Bridge at Yokohama, which was opened to traffic a few months ago. It is a three-span highway bridge with a double-track street railway and has a width of 78 ft. Originally a single arch of 120 ft. span was proposed, but owing to the low elevation of the banks, the height of the crown of this arch would have involved undesirably steep grades. The length of the bridge was divided therefore into three spans, the central span being made as long as possible to accommodate the river traffic. This arch has a clear span of 60 ft. between the piers, with a radius of 78 ft. and a rise of 6 ft. 6 in. above the springing line; the two side arches are of 25 ft. span, 19 ft. radius and 6 ft. 2 in. rise. A view of the completed structure is shown in Fig. 1, while Figs. 2 and 3 are a sectional elevation and cross-section, respectively.

It will be noted that the rise is almost as great in the short spans as in the long span, this feature being adopted in order to give ample headway for boats, so that all the spans could be available for navigation. This condition causes an unbalanced thrust at the piers, to take care of which the piers were made heavy and thoroughly reinforced to take care of bending stresses. The heavy foundations for the piers were placed eccentric to the center line of the pier proper, in order that the resultant line of thrust might be more nearly central with the base of the pier. This is shown in Fig. 2. In spite of this arrangement, however, a certain amount of the thrust had to be carried through the smaller arches directly to the abutments. For this reason, these arches were made extra heavy, and were well reinforced to take care of the negative bending moments resulting from this thrust. The reinforced-concrete abutments are of the buttress type, with buttress walls 4½ ft. apart, connecting the face wall and base.

The steel reinforcement is of the Kahn system, supplied by the Trussed Concrete Steel Co., of Detroit, Mich. The arches have trussed bars in both the intrados and extrados, the diagonal shear members of these bars crossing each other so as to form a lattice work of reinforce-

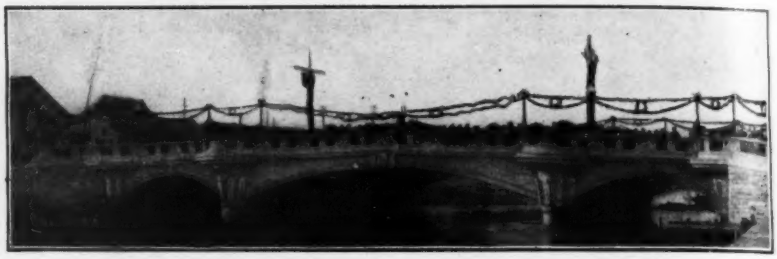


FIG. 1. YOSHIDA BRIDGE AT YOKOHAMA (THE FIRST REINFORCED-CONCRETE BRIDGE IN JAPAN)

(Dr. Ishibashi and S. Awano, Engineers.)

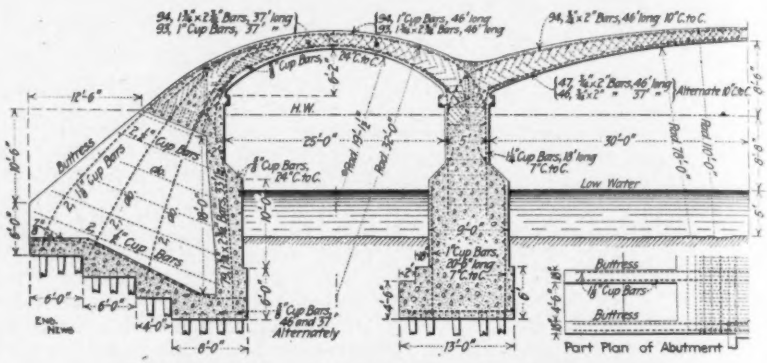


FIG. 2. HALF SECTIONAL ELEVATION OF THE YOSHIDA BRIDGE

(The eccentric position of the pier footings is to take care of unbalanced thrust.)

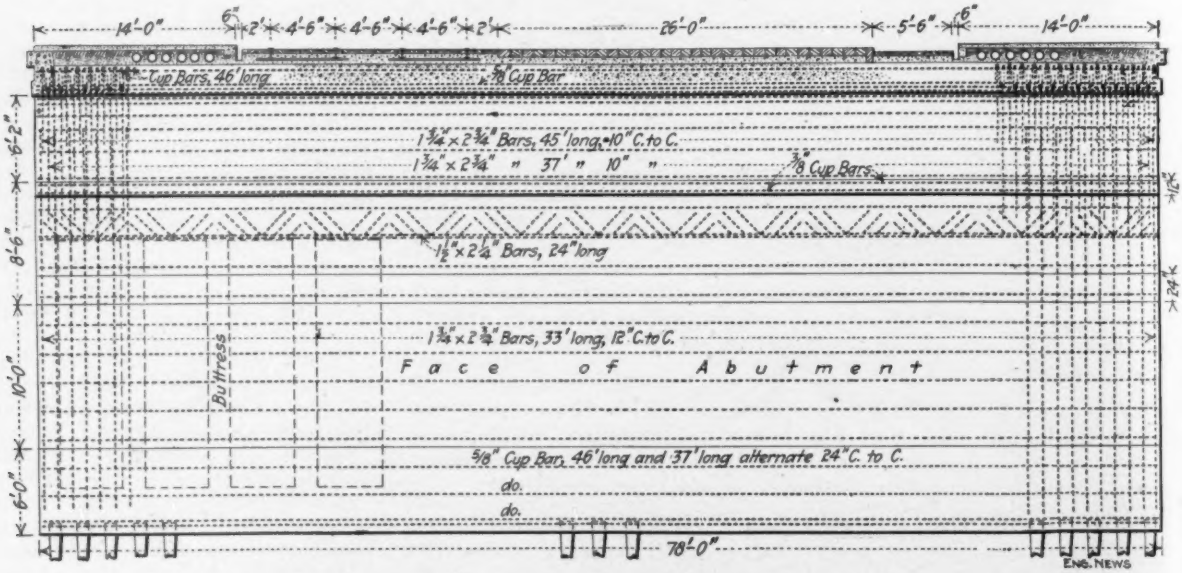


FIG. 3. CROSS-SECTION OF THE YOSHIDA BRIDGE, AT YOKOHAMA, JAPAN

(Section through side span.)

ment in the body of the arch. These longitudinal bars are extended into the piers and abutments. Transverse bars are laid also in both the intrados and extrados. The bridge has pile foundations, the bearing power of the piles being tested before the concrete work was commenced. All falsework was of bamboo poles lashed together.

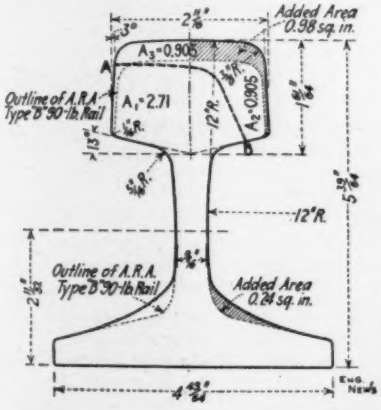
It will be seen by Fig. 1 that the bridge has a decorative treatment, and at each pier and abutment there is a circular balcony forming an extension of the sidewalk and balustrade, and supported by a flaring head on the pier and abutment. This view shows the decorations for the opening ceremony, these being festooned between the ornamental lamp posts. On the bridge also there are two ornamental poles for the trolley wires.

The bridge was designed under the direction of Mr. Ishibashi; and Mr. Awano was the resident engineer in charge of construction. Owing to the importance of the structure and the novelty of its construction (in Japan) special care was taken in working out the design, and tests were made of the bearing properties of the subsoil and of the strength of the reinforced-concrete work.

Letters to the Editor

A New Section for Manganese-steel Rails

Sir—The rail failures of the past year have shown that the present sections of rail are not designed with sufficient strength to withstand the strain of the increased weight of equipment and higher speed. The great majority of failures have started on the edge of the flange or in the fillet between the base and the web. It is obvious that this part of the rail must be strengthened.



	Sq.in.
Head	4.60
Web and flange	5.72
Total	10.32

DESIGN FOR 105-LB. MANGANESE-STEEL RAIL

(Manganese Steel Rail Co.)

As rails with a heavier base and higher fillets between web and base are now being generally considered, and as such sections involve increased outlay without any increase in metal available for wear, the section shown below is offered for discussion. This section is built up from the American Railway Association Type B 90-lb. T-rail. An addition of 5 lb. per yd. was made between the base and web as shown. Such addition, we are advised, has for some years been the practice on several foreign roads, with good results. It involves an increase in the whole section from 90 lb. to 95 lb. The novelty of the present suggestion, therefore, consists of the addition of more available head metal, as shown in the space between the solid line at the top and side faces of the head and the inclosed dotted line (which conforms to the contour of the 90-lb. head). This added head metal is 10 lb. per yd.

The heavy dash line *AB* indicates the top contour of head on a worn outer curve-rail when the wear has reached 25% of the head metal. The metal removed at this stage of wear on the 90-lb. Type B rail is 9 lb. per yd., and, as it would seem that the rail with the higher head could be worn down to the line *AB* before the rail would need to be discarded, the head metal available for wear on the proposed section would be 18 lb. per yd., or twice as much.

The figures in Table I below may be assumed as the basis for comparative cost calculations. The ratios given for the comparative life if manganese steel, bessemer and openhearth have been proven, and may be accepted as correct.

Assuming the figures in Table I and allowing interest to be compounded for a period of 12 years, the cost of maintenance per mile per year on the two sections and with the different metals would be as shown in Table II. The ratio between the different cost figures would, of course, vary somewhat, according to the life of the openhearth rail, which will in turn depend upon the severity of the service.

TABLE I

Life of 0.75% carbon openhearth rail	1 year
Cost of openhearth rail per ton	$32.00
Cost of bessemer rail per ton	$28.00
Cost of manganese-steel rail, per ton	$80.00
Interest rate	5%
Relaying	$3.00
Scrap value of openhearth or bessemer per original ton	$11.50
Scrap value of manganese-steel per original ton	$15.00
Life of manganese-steel rail, compared to 0.75% carbon openhearth	6 to 1
Life of manganese-steel rail, compared to bessemer	11 to 1

TABLE II

AVERAGE ANNUAL MAINTENANCE PER MILE FOR A PERIOD OF TWELVE YEARS

Bessemer rail, 90-lb. A. R. A. "B"	$7078.38
Openhearth rail, 0.75% carbon, 90-lb. A. R. A. "B"	4700.77
Openhearth rail, 105-lb. proposed section	2867.65
Manganese-steel rail, 90-lb. A. R. A. "B"	2631.71
Manganese-steel rail, 105-lb. proposed section	1814.97

W. S. POTTER, of
Manganese Steel Rail Co.
30 Church St., New York City,
Apr. 30, 1912.

Big Ships and High Buildings

Sir—Some of the recent accidents to very fast trains and, more recently, the terrific ocean tragedy of the "Titanic," point, it seems to me, to the fact that it takes some terrible tragedy like the Iroquois Theater fire or the "Titanic" disaster to make us appreciate the things which ought to be obvious, and which would be obvious to anyone if they stopped to consider.

It appears to me that the next great tragedy which is to be enacted will occur with some one of the high office buildings such as are being built in New York City today. A trifling earthquake, however remote the possibility; a terrific windstorm or cyclone; a fire in some one of the lower stories which would cause a panic in the building, or—worse yet—cause the buckling of some of the columns, it appears to me are among those possibilities.

Aside from this, these very high buildings are producing traffic conditions which our "one-story" streets and subways are totally unable to meet. Shall we go on blindly, regardless of the most obvious possibilities, and shall we let individuals having too little regard of these possibilities simply vie with each other in exploiting the public?

F. A. DELANO.
514 Western Union Bldg., Chicago,
Apr. 26, 1912.

[In the San Francisco earthquake, the steel-frame skyscrapers and their occu-

pants suffered much less than the buildings of ordinary brick and stone construction or even monumental buildings like the City Hall and the fine stone structures at Leland Stanford University. The danger of panic in a high building and of fires which may make a holocaust of the occupants of one or several floors, perhaps with little injury to the structure itself is, however, a real one. One needs but to recall the Asch Building disaster of March, 1911, to have concrete evidence of this.—ED.]

Tensile Tests of Cement Briquettes Exposed to Fresh Water, River Water, and Artificial Sea Water

Sir—The City of New York through its Department of Docks and Ferries has constructed about eight miles of bulkhead wall along the shores of the island of Manhattan and in other parts of the harbor, the base blocks of which consist of mass concrete molded in forms in air. After becoming sufficiently hardened, these blocks are placed upon the foundations prepared for them, where they remain exposed to the action of the salt water of the North and East Rivers, or the sea water of the outlying sections of the harbor.

The concrete work thus produced by the city is now in a very satisfactory condition notwithstanding the alarming reports on the condition of concrete harbor works similarly exposed to sea water which appear periodically from many other parts of the world. A year ago the writer ordered a series of tests which were performed at the testing laboratory of the Department of Docks and Ferries under the direct supervision of William Lansing, Jr., Assistant Engineer, by a careful manipulator with long experience in cement testing. The results of these tests are submitted as they will no doubt interest some of your readers, and add in a small way to the literature upon this subject. A series of tests was made on briquettes composed of neat cement and one part cement to three parts of standard quartz sand, for periods of time ranging from seven days to twelve months (of 28 days each), and the results of these tests have been plotted and are shown on the accompanying diagram.

Seven different brands of well known and largely used American portland cement, and one of German manufacture were used in making the briquettes, and the lines shown on the diagram represent the average breaking strength of all the briquettes. The results of each particular brand vary only an insignificant amount from the average lines, as plotted. All briquettes were made with fresh water, and upon completion some were exposed to fresh water, some to the ordinary salt river water, and some to artificial sea water, consisting of

Sodium chloride.....	($Na\,Cl$)	11.5%
Magnesium chloride.....	($Mg\,Cl_2$)	1.4%
Magnesium sulphate.....	($Mg\,SO_4$)	0.9%
Calcium sulphate.....	($Ca\,SO_4$)	0.6%
Water	(H_2O)	85.6%

The briquettes of neat cement exposed to the action of sea water show considerably greater strength than those exposed to the action of fresh water or river water; and those exposed to the action of fresh water show the lowest tensile strength. The difference of those exposed to the action of fresh water and sea water amounts to about 300 lb. per sq.in. The maximum strength appears to have been attained at the end of five or six months. In the case of the briquettes composed of one part cement to three parts of sand, just the reverse is the case; the briquettes exposed to the

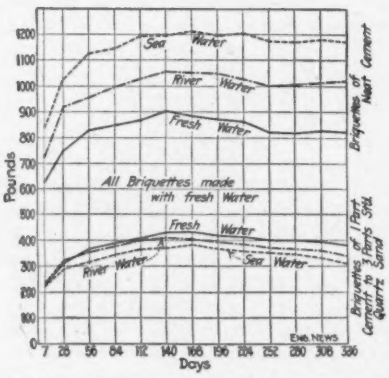


DIAGRAM SHOWING AVERAGE RESULTS OF TENSILE TESTS OF NEAT AND 1:3 BRIQUETTES MADE WITH VARIOUS PORTLAND CEMENT AND EXPOSED TO FRESH WATER, RIVER (SALT) WATER AND ARTIFICIAL SEA WATER

action of fresh water showing greater strength, and those exposed to the action of sea water the least. The difference in strength, however, amounts to only about 50 or 60 lb. per sq.in. The maximum strength appears to have been attained in from five to six months, as was the case with the briquettes composed of neat cement.

From the downward direction of the curve, at the end of the twelve months, it would indicate that longer time tests would be advisable, and these will be undertaken at once.

In the tanks containing the briquettes immersed in river water and sea water, there was a precipitate covering the entire bottom of the tank, which in the case of the river water was 2½ in. thick, or 46% of the volume of the water, and in the case of sea water ¼ in., or 3% of the volume of the water. The precipitate in the river water was of a light amorphus nature, which, when placed in a bottle with water and shaken, resembled milk in color and consistency.

The results of the tensile tests are the average of breaking five briquettes at each test.

Pats of the various brands of cement were made, using both fresh and river water, and exposed in tanks to the action of fresh water, river water and sea water; and at the expiration of the 336-day period, none of them showed either cracks, distortions, discolorations or disintegration.

CHAS. W. STANIFORD, Chief Engineer,
Department of Docks and Ferries.
New York City, Apr. 18, 1912.

The Special Joints on the 63-in. Submerged Steel Outlet Sewer, Cleveland

Sir—Referring to the note from Mr. Fenkell (p. 848 of our issue of May 2, 1912) regarding the joints used in laying the 63-in. steel submerged outlet pipe for the intercepting sewer system of Cleveland, a description of which appeared in ENGINEERING NEWS of Mar. 28, 1912, I beg to state that no claim for originality was made for these joints, and the term "special submarine joint" was used in the brief description merely to indicate the use of a joint every 150 ft., different from the regular flange joints used every 50 ft.

Before the specifications for this work were drawn up, in 1906, a copy of the Erie report was received from the Commissioners of the Erie Water Works, upon request, it being known that they had laid a pipe of about the size of our proposed outlet pipe, under similar conditions, and that we would appreciate such information as they could give us regarding the methods adopted by them in laying their intake pipe. It was not known then that the joint was original with Mr. Fenkell.

The entire article, however, was only a brief description of methods used, and no claim for originality was made in the use of the hydraulic shield, wooden cants, or other special devices used on the work.

The joint designed by Mr. Fenkell has given satisfactory service for our purpose.

J. M. ESTEP,

Notes and Queries

C. A. P. Turner, Minneapolis, Minn., asks us to note that the patent decision rendered against him in his suit against Moore & Scriver for infringement on his "mushroom" reinforced-concrete flat-slab patents (Engineering News, Apr. 18, 1912, p. 750) was brought on the claims of his narrow patent No. 985,119 and not on his broader patent No. 1,003,384. On this ground an appeal has been made.

Issued Weekly by the
Hill Publishing Company
JOHN A. HILL, Pres. ROB'T MCKEAN, Sec'y.
505 Pearl St., New York.

Monadnock Bldg., Chicago.
6 Bouverie Street, London, E. C.
Unter den Linden 71—Berlin, N. W. 7.

CHARLES WHITING BAKER, Editor.

Subscriptions payable in advance, $5.00 a year for 52 numbers, including postage in the United States, Mexico, Cuba, Porto Rico, Hawaii, or the Philippines, $7.00 in Canada.

To foreign countries, including postage, $9.00 or its equivalent, 36 shillings: 36 marks: or 45 francs.

Notice to discontinue should be written to the New York Office in every instance.

Advertising copy should reach New York Office by Thursday of week before date of issue.

Entered at New York Postoffice as mail matter of the second class.

CIRCULATION STATEMENT

During 1911 we printed a total of 931,958 copies.

Of this issue, 19,000.

None sent free regularly, no returns from news companies, no back numbers kept beyond current year.

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Engineering News

Cleaning the Augean stables was a light task compared with the municipal housecleaning which is going on at Philadelphia. The latest cleaning work reported is the abolition of piggeries within the city limits. May 1 was the day set for the departure of the last hog, but weeks before that date it was estimated that already 15,000 had grunted their last good-by to the city of brotherly love—and pigs. The banishment of the pigs has been all the more difficult owing to the fact that there was a profit to many in gathering garbage from hotels and restaurants for feeding to swine. There is more or less serious talk of adding this refuse to other city garbage and treating it all in a municipally owned garbage-reduction works, from which the city, instead of private parties, would derive whatever financial benefit accrues. We presume that other and more important sanitary and economic reforms will take precedence over this one.

The relative value of crushed stone and gravel for concrete aggregate was a favorite subject for discussion some years ago when the nature of concrete was just beginning to be studied extensively and very decided views were held by the adherents of each material. Of late, however, the ardor of the controversy has considerably abated and the average engineer is willing to accept either crushed stone or gravel if the one selected is satisfactory in every respect. Nevertheless a number of standard texts on concrete contain very definite statements to the effect that tests show that broken stone concrete gives higher compressive strength than concrete made from gravel, the mixes by volume being equal. It is interesting to note, therefore, that the recently published tests of reinforced-concrete beams and concrete cubes and cylinders made by the Structural Materials Laboratory at St. Louis, show a slight superiority in compressive strength of gravel concrete over broken granite concrete and a considerable superiority of the gravel concrete over crushed limestone concrete.

We do not wish to reopen this old controversy nor to claim superior qualities for gravel as a concrete aggregate; we merely call attention to this record as proof of the correctness of the now generally accepted equality of the two materials. The fact is that the individual aggregates, their composition, source and manner of preparation, have more to do with the strength of a concrete made from them than the general question of whether they are gravel or crushed stone. Any specification that specifies one class to the exclusion of the other class, merely because it belongs to that class, is not a definite or a satisfactory specification.

A Summary of Government Dredging Work

Anyone reading the uplift literature circulated by the promoters of huge appropriations for waterway improvements might suppose that the United States government at the present was doing little for the betterment of waterways used by commerce. It is likely that comparatively few, even among engineers, realize the enormous magnitude of the work which the government is now carrying on for the improvement of rivers and harbors.

Elsewhere in this issue there is presented a summary of the work which was done in the year ending June 30, 1911, by a part only of the dredging fleet owned by the United States government. It will surprise many, we are sure, to learn that the fleet owned by the government for use in waterway improvement work comprises no less than 855 vessels of all classes. Of dredges alone there are 137. There are eight vessels in this fleet of dredges which have cost in the neighborhood of $400,000 apiece. In sea-going hopper dredges and hydraulic pipe line dredges alone, the government has an investment of about nine million dollars. The amount of material excavated by these two classes of dredges last year was about five-sevenths as much as the total contents of the great Culebra cut on the Panama Canal, the largest piece of excavation in the world.

Besides this huge volume of excavation, there was the work done by the government fleet of bucket and dipper dredges, 61 in number, and the great volume of dredging work performed for the government by contractors.

A still better idea of the vast magnitude of the work being carried on in river and harbor improvement may be gained by considering that the fifty odd million cubic yards of dredging done by the government's hydraulic dredge fleet last year, cost in the neighborhood of three million dollars and that the government is spending in the neighborhood of ten times this sum annually on river and harbor improvement. We respectfully commend these figures to the careful consideration of the vociferous writers and speakers who continually allege that the government is wholly neglecting to improve and develop the natural waterways of the country.

Contract Prices of New York State Road Work

We published last week a list of contracts for highway construction in New York just let by the New York Highway Commission. The total amount of these contracts was $4,178,000, and they covered 331 miles of highway. The average cost of construction, therefore, for the entire work is $12,620 per mile.

An examination of the table (on pp. 266, 267 of the Construction News supplement in our last issue) shows a very great variation in the rate per mile at which the contracts are let. There is one 12-mile stretch near Horseheads, N. Y., which is to be built for a little less than $3000 per mile; the next lowest is a six-mile road near Bath, costing $5451 per mile, and there are half a dozen more contracts at rates under $9000 per mile. On the other hand there are two contracts for short stretches of road a mile and a half in length, on which the cost is above $49,000 per mile, and there are seven other contracts at rates above $25,000 per mile.

Some of these contracts call for street paving apparently rather than highway construction. In other cases the contracts include not only the road surfacing, but such improvements as storm sewers, catch basins, curbs and gutters. This raises the question why the state should contract for curbs and sewers in towns or villages. For a reply, it is necessary to turn to the very involved highway law under which the road work of New York State is now being done. These statutes provide for roads built entirely by the state funds, and also for county roads, 50% of the cost of which is paid by the state, 35% by the county and 15% by the town. Recent amendments of the statutes permit a county and town to pay 2% and 1%, respectively, of the cost of a road for each $1000 of assessed valuation per mile of highway, in place of the figures of 35% and 15%. This 2% and 1% has proved to be much less than the 35% and 15%, so that the actual proportion of the state has risen to some 65%. The law provides also for the maintenance and improvement of town highways, the state paying a sum equal to from 50 to 100% of the total local road taxes. Moreover, the law defines a "highway" as including "culverts, sluices, drains, ditches, waterways, embankments, retaining walls and all bridges having a span of 5 ft. or less," and provides that the state may build county highways through villages and cities and construct them in accordance with local desires, charging the city or village with the excess cost over that of the ordinary county highway. It is evident then that the contract prices alone do not show what the state is getting for its expenditure. Only careful accounting can determine whether, on one hand, the state is paying for town improvements like curbs and gutters or, on the other hand, the town is paying for a better roadway than it needs merely to facilitate the passage of county traffic along its streets.

Better Methods Needed for Handling Lifeboats

We urged last week that public attention ought not to be concentrated on the lack of sufficient lifeboats on large passenger vessels to the exclusion of other equally important lessons of the "Titanic" disaster. If it is important that lifeboats enough be carried on board steamers to accommodate all the ship's company in case of emergency, it is certainly of equal importance that the method of carrying and lowering such boats when the emergency comes shall be the best that ingenuity has devised. It has been stated that the "Titanic" was equipped with one of the latest improved forms of boat davit and that it was due to this fact, in part at least, that practically the entire complement of boats was loaded and launched with the loss of but one or two boats and with no casualties to life.

It is indeed no small engineering feat to provide machinery powerful enough to hoist a boat big enough to carry 60 to 70 passengers, bring it out over the vessel's side, permit it to be loaded to its capacity and safely lower it to the water's surface 80 ft. or more below. This work has to be done, moreover, under emergency conditions when high excitement and semi-panic may exist. It may have to be done when the vessel is badly listed and when the ship is rolling and high seas are running. It is little short of marvelous that even with the favorable conditions of wind and weather the new undrilled crew of the "Titanic," with practically no previous boat practice, should have been able to so safely and successfully launch the vessel's boats and fill them with those who might be saved.

It is of interest to note that only a fortnight previous to the "Titanic's" loss, an important discussion took place at the meeting of the Institution of Naval Architects in London on lifeboat installations for modern vessels. A paper was read by A. Welin, inventor of some of the most recent types of davits for handling lifeboats*, in which he proposed that to obviate the danger and difficulty inherent in launching boats from the great height of the top deck, the lifeboats on large steamers should be carried on one of the lower decks. This would greatly reduce the difficulty and danger in launching them, especially when the vessel was listed as a result of collision. It was stated that the plan is under consideration for adoption on one of the largest ships in the world.

A statement in the discussion, as reported in London *Engineering*, of Apr. 12, is that "though shipowners would kick against occupying one of the lower decks of the steamer with lifeboats, they would object wherever they were placed. Boats were much disliked as they occupied space and earned no money."

However true this statement may have been when it was made, it is safe to say that following the loss of the "Titanic," shipowners will give attention as they never have before, not only to the provision of a greater number of lifeboats but to means whereby they may be safely lowered under the difficult conditions which are only too apt to prevail at the time the boats are needed.

*The quadrant types were described by Mr. Welin before the Society of Naval Architects and Marine Engineers, at New York City, Nov. 21, 1907; see Eng. News, Nov. 28, 1907, p. 591.

How Much of the High Typhoid in New York Cities is Due to Impure Water Supplies?

The good work of studying the relation between the character of the water supplies of the cities of New York State and the typhoid death rates of those cities which was begun some years ago by the State Department of Health has been continued and the results for 1911 are published in the March *Bulletin* of the department. The average typhoid death rates for the ten years 1900 to 1909 are presented, as well as the total typhoid deaths and the corresponding rates for 1910 and for 1911. The source of water supply for each city is stated, with remarks on the character of many of the supplies. Since an even more detailed table for 48 of these 49 cities, covering the ten years 1899 to 1908, was printed in our issue of May 7, 1909, we shall give at the present time only the department's later summary according to sources of supply, adding a column of our own to show the number of cities in each group:

TYPHOID DEATH RATES IN 49 NEW YORK CITIES, SUMMARIZED BY SOURCES OF WATER SUPPLY

Water supply	No. of cities	Typhoid deaths per 100,000 Average, 1900-1909	1911
Unfiltered water from streams and reservoirs*	13	26	10.0
Unfiltered lake water	7	26	18.6
Filtered water from streams and reservoirs	7	27	23.0
Well or spring water	7	32	25.0
Filtered river water	7	37	25.0
Unfiltered river water	8	56	64.0

*Supplies from relatively small streams, generally if not always impounded in reservoirs, and thus having more or less benefit from storage. New York City is included in this list; but a minor part of its supply is from wells and a small percentage of the surface supply is filtered.—Ed.

The department states that the water supplies are "the chief cause of typhoid"

and again: "The law still holds that the prevalence of typhoid fever is a measure of the purity of the water." It promises to present in a later *Bulletin* the distribution of typhoid by seasons and also to present "characteristics of its prevalence not exhibited" in the data before us. We sincerely hope it will be able to make a broader study of the cause of these high typhoid death rates than it did in 1909 or than appears in its March *Bulletin*. The desirability of this was urged in the editorial discussion, already mentioned, which we published on May 27, 1909. We fear, however, that the necessary data for settling some of the questions raised by the summary which we print are lacking and never will be available until many of the local health departments of the state are reorganized on a more efficient basis. There can, however, be no doubt but what the persistently high typhoid rates which prevail in some of the cities having notoriously bad water supplies are due to the badness of the water. But we believe that other things than water supplies need improving in many of the cities and that these, too, should be held up to public view by the State Department of Health. As we pointed out in the editorial already mentioned, the department doubtless lacks funds for getting at all the facts, but it seems likely to continue to lack them unless it makes the need known.

One thing is certain: If the high typhoid rates reported are chiefly due to impure water, then a large percentage of the water supplies of New York are bad indeed, and vigorous measures for their improvement should be taken immediately.

Accidental Electric Shock from Fixtures on Alternating Current Circuits

A month ago, at Rutland, Vt., a young attorney, prominent in the affairs of his state, attempted to turn on an electric lamp while in his bath and was instantly killed by electric shock.

The prominence of the victim and the shock to his family and wide circle of friends make this a fitting time to call public attention to this too frequent source of danger and death. Electrical engineers well know that cases of fatal shock from electric-light fixtures are all too common, and it is hardly creditable to the profession that so little has been done in a systematic way to eliminate or reduce this danger.

The traditions of early days in the electric-lighting industry have too long survived when every case of accidental electrocution was hushed up as quickly as possible, lest it should hurt the business.

Even now, we doubt not, there will be those who will aver that such accidents are so rare that it is not worth while to do anything whatever to prevent them. But are they so rare?

In our search for statistics of death by electric shock we sought aid from the accident records of the State Public Service Commissions. There are now 26 states, provinces and cities on this continent in which such Commissions have been organized; but in the reports of none of these did we find any records of casualties by accidental electric shock in such form as to shed light on the question at issue.

Fortunately, however, the electric-lighting industry itself has become aroused to the gravity of this matter and has undertaken the collection of statistics.

In 1910, a committee of the National Electric Light Association sent out inquiries for information on the extent of these accidents among its members, and received replies from 285 concerns. This number was 36% of the member companies, and about 3½% of all the light and power companies in the United States, Canada and Mexico; these concerns, it is claimed, however, represented 90% of the capital invested and probably 80% of the total number of persons supplied with electric current.

The committee found, in the few years' records which they could reach, 64 fatal and 42 serious accidents of the type in question.

If the same proportion of accidents to customers held throughout the industry, the number of fatalities would have risen to 220 and the cases of severe injury to 145. It would not be surprising, if the truth could be ascertained, to find the proportion increased among the small concerns not represented in the National Association, since that association includes what we may call the most progressive concerns, the equipment of which would be the best and the accidents fewest.

Some would claim, and we think with a little merit perhaps, that the number of such accidents would increase with the number of companies rather than of customers, because of the greater chance of damage from lightning in the much greater area covered. If that were the true situation, we could expect no less than 1500 fatal and 1100 serious accidents of this nature would have occurred up to 1910. Such a great toll could not have failed to attract attention and probably was never realized. Giving all considerations due weight, it seems probable that the fatalities might have risen to 250 and the lesser accidents to 170.

These figures are surprising, but a careful study of the evidence on which they are based tends to show that they are conservative rather than liberal estimates. It is evident, therefore, that there exists a grave danger to life on all of those electricity-supply systems where latest knowledge has not been used to prevent trouble. Equipment and methods have been highly developed so far as efficiency, reliability, and cost of service are concerned, but the repetition of these accidents is evidence that complete safety has not been maintained.

In 99% of the town systems, there is a high-voltage primary-distribution line, low-voltage lines leading into the houses, and transformers forming a magnetic, rather than an electrical, link between the two circuits. In times of accidental ground on the high-potential distribution lines, the safety of all the customers of that electric-lighting company depends on a comparatively thin section of cotton, paper, porcelain, mica, or oil between the high- and low-voltage winding of the house transformers. This may be ruptured by lightning discharge, by some accidental contact with long-distance transmission line of extreme voltage or perhaps by some obscure surge phenomenon, the liability of which increases as the magnitude of the supply company's network increases. There is also chance of accidental direct contact, brought by falling wires, between the high-tension local-distribution line and the low-tension house wires.

It is a most natural act to grasp a steam, gas or water fixture while reaching to an incandescent lamp. Any unwary person may thus form a partial short circuit on the higher potential wires in case of trouble. The result is usually death, though sometimes fortunately only bad burns and severe nervous shock.

This danger from contact with abnormal potential on house circuits has been recognized by electrical engineers since the earliest days of the alternating-current system. They have experimented with all sorts of automatic devices to cut off connections when a dangerous voltage was approached or to connect with ground the lines in trouble. None of these schemes have proved reliable, nor should much be expected of them since they are not regularly in operation and are called on to act only perhaps years after installed and when deteriorated from lack of attention. They necessarily involve a delicate construction not to be compared with that of safety valves and such rugged mechanical devices. Grounded metallic shields between the primary and secondary coils of the transformer have been tried but found neither reliable nor safe. It is rarely possible to secure a good ground with the transformer on a pole in an open lot; and the metallic shield between primary and secondary windings weakens the insulation in case of lightning and surge disturbances, so that under some conditions the shield may be a menace rather than a protection.

It is now widely accepted as good

practice to establish a permanent connection between one low-tension house-supply lead and ground. Then no higher voltage can appear inside the house than that of the normal supply, which is harmless except to abnormal individuals. Any connection between the high- and low-potential circuits, as the result of transformer troubles or accidental contact, is short-circuited by the ground connection; if any current flows it merely blows fuses somewhere beyond the house, cuts off the current and warns of the trouble by failure of the supply.

There has been a natural hesitation in thus grounding the house lines, since the small supply voltage may then be received in handling service fixtures by unwary persons holding on to pipes or standing on wet floors. This may be unpleasant in some cases, but it can be dangerous only by reason of a weak heart and impaired nervous system.

An additional safeguard, which can be adopted in many cases, is to locate fixtures, wherever possible, so that no one can touch them and at the same time make contact with any part of the plumbing or heating apparatus. Where that is actually impossible and for lamps in basements or other places where wet floors or concrete floors may exist, porcelain-covered sockets and protected remote-control switches may be used. It might be worth while to insert insulating joints in the gas and heating mains; such a move would be ineffective for water pipes because of the continuous column of water in the pipes bridging the joints.

It is necessary that the greatest pains be taken to have the ground connection perfect, else it is ineffective. For absolute reliability, a heavy wire should run to city water pipes. This is not possible in many instances since water-supply companies and even municipalities forbid it under the mistaken idea that it might injure the pipes by electrolysis. Electrolysis, however, is caused by direct-current leakage from pipes and no evidence has been found up to the present of appreciable deterioration of buried pipe where alternating current passes to ground. Moreover, the pipe in this case is only an emergency conductor and currents flow through it only at extremely rare intervals. Further opposition of water companies to such use of their pipes permits a continued danger to life and is as inexcusable as it is senseless.

Grounding secondary circuits has been advocated by the National Electric Light Association and American Institute of Electrical Engineers for many years, but it has been too infrequently carried out because of ignorance and prejudice of company and municipal officials. This prejudice survives even today, and makes this safeguard impossible in many places where enlightened operators are desirous of making the change. It is probable that some of this prejudice was fostered by the fire-insurance underwriters' early action in not permitting grounding, as a result of their mistaken notion that it increased the fire hazard. But the underwriters saw the error of their views and for many years have gone to the extent of their power in advocating grounded secondaries. Their power is limited to recommendation since the hazard is rather one of life than of fire. As recently as Mar. 12 of this year, a committee of the American Institute of Electrical Engineers, the Association of Edison Illuminating Companies, the National Electric Light Association, and the National Inspectors' Association voted unanimously to recommend to the electrical committee of the National Fire Protection Association that it make mandatory in the National Electric Code (rules governing the installation of electric wires) the grounding of secondary circuits of 250 volts and less.

In the long run the only mandatory power lies in the acts of legislatures and the orders of public-service commissions. It is the duty of legislatures, commissions and councils to demand protection. Engineers everywhere should exert every influence with these bodies and they may recommend grounding with full assurance that this is the greatest measure of relief which our present knowledge affords.

The U. S. Navy's Lead-Lined Piping*

By R. D. Gatewood†

Steel tubing lined with lead is used on shipboard in the U. S. Navy for all salt-water pressure piping over 1½-in. in diameter. It has been found superior to all other pipings for this use in resisting corrosion.

Some account is given of the steps leading to its introduction, followed by a description of the method of manufacture and notes on practical points to be observed in its use.

*Abstract of an article in the December, 1911, "Proceedings" of the U. S. Naval Institute.

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There is probably nothing on a modern man-of-war that causes as much constant trouble both for the ship's force and the navy yard as the various piping systems. Before describing the process which is believed to be the last, or rather the latest, word toward the removal of the difficulties, it may be of interest to review the history of the corrosion on board ship of the non-ferrous metals used in these systems together with the results of some of the leading investigators of this subject.

Instances are on record in the early 18th century of serious corrosion of the iron fastenings of vessels sheathed with lead, which sheathing was accordingly abandoned and copper, with copper fastening, used in its place.

Occasional cases of serious corrosion of copper sheathing still arose and Sir Humphry Davy about 1820 conducted some experiments along preventive lines, the results of which were to show that the sheathing was perfectly protected when the ratio of its area to that of the protective metals, iron or zinc, was approximately 1000 to 1. For a while this form of protection was used but it was found that the iron or zinc required very frequent renewal and the sheathing thus protected was much more easily fouled by marine growth than when not protected, and the scheme was abandoned.

In 1832 Muntz metal replaced copper as sheathing, and for a long time there was comparatively little trouble with sheathing or piping except in isolated instances and many officers now living can well remember the pride with which the chief engineers of the old navy would exhibit the highly burnished copper piping in the bilges at Sunday captain's inspection.

When iron and steel came to be universally adopted for hull construction a new problem arose. The copper pipes, bilge water, and steel floors and angles and the shell formed an excellent electric battery and very serious corrosion of the electro-positive steel resulted, as a consequence of which copper pipes in the bilges were reduced to a minimum. There is an instance of a vessel that was run ashore to prevent her sinking from a hole in her shell caused by a copper strainer in a bilge suction pipe.

Another period of comparative immunity ensued until the general introduction of electricity on shipboard, and from that time until the present the trouble has been constantly increasing. The increased steam pressures occurring about the same time were frequently blamed for some of the difficulties, but in the light of the modern electrolytic theory of corrosion they may safely be eliminated as a cause.

The corrosive action occurs in the valves and piping of the fire, flushing,

drainage and fresh-water systems, and in the tubes of condensers and distillers. It takes several characteristic forms, among the most common of which are the following: Sometimes the pipe is eaten through in pin-holes; sometimes the holes are as large as ½ sq.in. in area with sharp, clean-cut edges as though punched; again they are pitted and honeycombed as though the pipe had a disease similar to smallpox; and still again there are long clean-cut longitudinal grooves in the tubes or pipes.

Corrosion is found generally at or near the bends, branches and valves. In the case of ordinary piping, the pits or holes are usually found in the top of the pipe and near the flanges while in condenser tubes about 80% of the pitting is found in the bottoms. Throughout the whole phenomenon the action has been found to be exceedingly erratic. Under conditions where corrosion has been found to be most virulent, cases are encountered which, while apparently similar in every respect, show most excellent states of preservation.

5½-IN. STEEL PIPE LINED WITH LEAD, SAWED OPEN TO SHOW INTIMATE CONTACT OF LINING

MEANS OF PREVENTION

A knowledge of facts established by observation of conditions under which pipe corrodes has produced many and varied recommendations as to the best means to be adopted to prevent corrosion, the most important of which, up to about 1905, are the following:

1. The use of the more electro-positive metals such as sheets or slabs of iron and zinc in direct metallic contact. Our navy uses zinc slabs inserted in zinc boxes installed in pipe lines from 30 to 40 ft. apart. The British use cast-iron freely. This they say is equally as effective on account of better metallic contact and being much cheaper and easier to insert, which they do by means either of cast strips and rods let in among the tubes in condensers or short lengths of cast-iron pipe in their piping systems.

2. The use of some form of coating to protect the surface of the metal from contact with sea water. One school of engineers and chemists advocated the use of non-metallic coatings, such as marine glue, varnish, lacquers, asphalt, porcelain and special forms of cement and wax. The best known form of this type was that called the Sabin process, named from its inventor Prof. A. H. Sabin, and used extensively for several years in our navy. The process was expensive, involving as it did very careful cleaning of the pipe, dipping into a patented hot liquid coating and baking in a special oven at a constant temperature with the pipe on end. Properly applied it did protect the piping but like all other coatings of this nature, it eventually cracked, scaled or eroded off with consequent clogging of small branches, valves and strainers; and further, in those spots where this occurred the corrosive action was materially intensified. For these reasons it was abandoned.

Another school advocated the use of metallic coatings and the most common were tin, lead, and tin-lead alloys. Until the present lead-lining process was perfected it was very difficult to apply these as entirely unbroken coatings free from pin-holes that seriously increase the corrosive action. Also they were not durable owing to their slight thickness.

3. It was recommended that the tubes or pipes be kept free from foreign particles by being constantly cleaned out, a proceeding as difficult as it is expensive.

4. Every care was to be used in preventing the access of stray electric currents to the piping or tubes.

TESTS OF LEAD-LINED PIPE

In 1906 experiments were made by Naval Constructor Stocker at the Norfolk Navy Yard that resulted in the present method of lead-lining. Experience with lead- and tin-lined pipe had been confined to screwed connections which had proven more or less unsatisfactory at the joints. Mr. Stocker's idea was to use seamless drawn steel tubing, galvanized to increase life against external corrosion, lined with an unbroken coating of lead forced by high pressure into intimate contact with the tubing, using throughout *only flanged joints*.

The success of this method was clearly demonstrated by electrolytic tests made in 1906-7 at the New York Navy Yard. These tests made it clear that lead-lined pipe is superior to all others for use on shipboard.

Further test was made by installing on the "Tacoma" in May, 1908, the same length of lead-lined pipe used in the New York Navy Yard tests. When examined a year later, it was found to be in perfect condition. This pipe had been in service for about nine months previous to the electrolytic tests.

Sections of lead-lined pipe were installed in 1906 and 1907 on the battleships "Kentucky," "Louisiana" and "Kearsarge" in the flushing pipes and fire mains. When removed and examined, after more than two years of use, the linings were found in excellent condition without sign of pitting or corrosion. The only evidence of use appeared to be a slight deposit on the surface of the lead.

In one or two of the sections examined it was noted that there was a slight tendency of the lead lining to draw toward the ends of the pipe, caused, no doubt, by the expansion and contraction of the pipe and lining.

DESCRIPTION OF MANUFACTURE

The marked superiority of lead-lined pipe having been thus clearly shown, it was adopted by the Bureau of Construction and Repair for all salt-water pressure piping above 1½ in. in diameter.

In repairing pipe lines brass, iron and copper piping may be lined, but for new work lap-welded steel casing (being much cheaper than seamless drawn tubing and equally satisfactory) is used. The inner surface of this casing is first made entirely smooth by a revolving chain mop on a flexible spindle attached to a pneumatic drilling machine. In the case of old piping, it is first cleaned and the pits closed by solder, wiped off smooth, and tested to a pressure of 300 lb.

All joints are made with flanges. When repairing a pipe having several dif-

ferent bends it is sometimes necessary to cut the pipe at the bend and fit flanges. The flanges are counter-bored to about ¾ in. from the face of the pipe, depending on the size of the flange, and about 1/10-in. deep to correspond with the thickness of the lead which is to be flanged over. Care must be taken that the edge of the casing over which the lead is bent is carefully rounded with a file.

The lead tubing used is about ¼-in. smaller in diameter than the pipe to be lined. Where bends are encountered it is necessary to pull the lining through. This is done by means of a rope led through the lead tube and then through a plug and secured by a knot. The lead tubing is then filled with dry sand, covered with tallow and pulled through by means of a machine consisting of a cylinder and piston, like that of a hydraulic hoist mounted horizontally at one end of a suitable frame. At the opposite end of the frame is a worktable to which is attached the pipe to be lined. The rope which pulls in the lining is passed through the pipe and secured to the piston, which is operated by air or water pressure at from 15 to 100 lbs. per sq.in. according to the needs. These machines were designed by J. T. Murphy, in charge of the pipe shop at the Mare Island Navy Yard.

While the lining is being drawn in, the outer tube is tapped with a wooden mallet. Bad bends are warmed slightly by means of a gasoline torch.

The plug, sand and rope are then removed and the lead examined by inserting a small electric lamp. If no defects appear the lead tubing is tapped over into the recess of the flange at one end of the pipe only. The other end of the lead pipe is sawed off about 6 in. longer than the pipe that is to be lined and a plug is fitted which permits the application of hydraulic pressure inside the pipe.

An air cock is provided in the plug to be opened while the pipe fills with water. When the air is all out, the cock is closed and the pressure is very gradually run up to 300 lbs.

The lead pipe becomes shorter under this pressure which is then removed and the lead fitted into the recess of the flange at this end of the pipe. The piece of piping is then blank-flanged at both ends and treated to a final pressure of 600 lbs. per sq.in.

Weight and Cost

In order to reduce weight, the lining should be no thicker than absolutely necessary. For sizes given in the following table, the weight and thickness there given have been found very satisfactory:

Outside diameter of pipe; in.	Weight of pipe; lb. per ft.	Thickness; in.
1½	2	0.088
2	3	0.099
3	4	0.088
4	5¾	0.095

The weight of such a system is about 5% less than that of a similar installation of copper piping. The cost is about 50% less.

Practical Points in the Use of Lead-Lined Pipe

1. The lead piping should be shipped from the works of the manufacturer very carefully packed on wooden mandrels to prevent crushing and distortion.

2. In the case of galvanized iron, wrought iron or steel tubing, the elbows, tees and other fittings are not lead-lined. It has been found necessary to lead line fittings under no circumstances.

3. Old pipe has been lined with lead where the radius of the bend was twice the diameter, but it is better practice to bend the tubing to be lined to about four or five times the diameter.

4. In replacing old copper pipe, it is much cheaper to reline the old pipe even though it is necessary to cut out the brazed flanges and substitute flanged fittings.

5. The process does not work well on pipe above 5½ in. on account of the crushing of the lead tubing when drawing it into the steel casing.

6. Lead-lined piping is not adapted to suction pipes. The suction pulls the lead from the walls of the steel casing.

7. Lead-lined piping is not adapted for use with very hot water or steam service, such as bottom blow piping for evaporators, on account of the large expansion in those cases.

8. Thickness of tubing thinner than that given in the above table should not be used. Considerable difficulty has been experienced on recent torpedo-boat destroyers in the attempt to save weight in this manner.

9. Special care should be used to give no crown to the lead where it is expanded over the recess of the flange. Unless a perfectly flush surface is thus obtained, it has been found that it setting up the joint the pressure comes on the lead, causing it to separate from the flange recess or to crack where bent over the steel casing.

10. Several cases of severe lead poisoning have occurred at Mare Island due to the milky oxide of lead on the outside of the tubing getting into the pores of the skin of the workmen and all the usual precautions should be taken to guard against this.

A Tide-Power Plant is at present being built in Germany. The location is between Husum and the island Nordstrand. Dikes are being constructed to form two reservoirs of 1500 acres and 2200 acres, respectively. The power equipment (turbine) is 5000 hp. The larger reservoir is the high-water basin, the other the low-water basin. The plant is intended to feed a power-distribution system in Schleswig-Holstein (Zeitschr. d. Oesterr. Ing. u. Arch. Ver., Apr. 12, 1912).

Proposed Joint Sewage Purification Works for Orange, Montclair and East Orange, N. J.

An interesting joint sewage-disposal works, intended to produce a high-grade effluent, has been designed for Orange, Montclair and East Orange, N. J., by Hering & Gregory, of New York City. The plans for the plant were submitted to the governing bodies of the three municipalities on Apr. 22.

The plans include a coarse screen, grit chambers, Imhoff settling tanks, sludge-drying beds, dosing tanks, sprinkling or percolating filters, a chemical house for preparing and applying hypochlorite of lime for disinfection, final settling tanks and laboratory. In addition, provision is made for a main sewer with branches. The estimated cost of the necessary sewers, large enough to serve the estimated population of 1940, and for disposal works sufficient until 1925, is $1,080,000. Of this sum, $690,000 is for the construction of the purification works and $125,000 for land for the works, making a total of $815,000 for purification alone; and $265,000 is for main and branch outlet sewers and right-of-way. The estimated population of the three municipalities in 1925 is 133,000, when it is calculated that the average quantity of sewage to be treated will be 17,000,000 gal. a day. The 20,200 ft. of main and branch sewers required to connect the present sewerage system with the works have been calculated for a population of 181,000, which is the estimate for 1940. Provision is made for treating the sewage from sanitary sewers, only. No pumping will be required.

The sewage of all three municipalities is now discharged into the Passaic River untreated: that of Orange and Montclair through a joint outlet, which also serves Bloomfield and Glen Ridge; and the sewage of East Orange through one of the Newark sewers, East Orange having long since abandoned the sewage-purification works which it put in operation in 1888.

The site for the proposed purification works is in the northwest corner of Belleville, about a half mile north of the Essex County Isolation Hospital, and not far from the Bloomfield boundary lines.

The effluent would be discharged into Third River, a small territory of the Passaic River, above the site chosen for the works. Third River has a drainage area of 8.6 sq.miles. The dry-weather flow at this point is at 3,500,000 gal. a day as an average for the driest month of an average year, with a possible minimum for a short period of only 750,000 gal. For the wettest month of an average year the flow of this stream is probably about 15,000,000 gal. The flood flow may range

from 140,000,000 to 1,000,000,000 gal. a day.

At present the Third River receives some trade wastes above and below the site chosen, but the stream is not in an objectionable condition and its waters are used for manufacturing purposes. Besides the small volume of the stream, the following reasons are given by the engineers for purifying the sewage to an unusually high degree:

> Below the site of the proposed works the river flows with a comparatively slow velocity for a portion of the way through a residential district. It is, therefore, not only desirable, but necessary, that the effluent, to be discharged into the river, should be clear and odorless, have no suspended matters that could settle in the ponds below, and furthermore, should be free from disease germs and from organic matters capable of putrefying and producing unpleasant odors. Such a well clarified and purified effluent can be produced by the proposed sewage disposal works and will have no undesirable effects either on the river itself or on the district through which the river flows.

The site for the disposal works ranges in elevation from 70 to 140 ft. above sea level, and affords facilities for an economical arrangement of the works. In outline the several parts of the plant, in the order of the sewage flow, are as follows:

SCREEN—This will consist of inclined flat bars, 2 in. apart in the clear.

GRIT CHAMBERS—There will be two of these, each divided into three compartments, 4x55 ft. in plan. Each compartment will be provided with stop planks at either end, for shutting off the flow when being cleared. When operating at the full 17,000,000 gal. capacity, and using four of the six compartments in the two chambers, the velocity of flow through the grit chambers will be about 1 ft. per sec., thus giving a detention period of about one minute.

IMHOFF SETTLING TANKS—There will be 16 Imhoff tanks. Each tank will be 30 ft. wide, 82 ft. long, and 26½ ft. deep from the water line to the bottom of the sludge chambers, with two settling chambers and four sludge chambers. Provision is made for reversing the flow of sewage through the tanks. At full 17,000,000-gal. flow it will take the sewage an average of three hours to pass through the settling chambers. The sludge chambers are designed to hold a 5½-month collection of sludge, but the detention period need not be so long.

SLUDGE-DRYING BEDS—The 16 sludge beds will have a total area of 45,000 sq.ft. and will be composed of graded gravel, having an average depth of 12 in., covered with coarse sand. The bottom of the beds will be graded into ridges and valleys. In each valley a 3-in. drain will be laid, connecting with 8-in. drains, which will connect, in turn, with the main underdrains from the sprinkling filters—thus the sludge-bed drainage will be disinfected. Small cars running on trackways will be used for hauling the dried sludge from the beds to the final point of disposal. The site comprises an abundance of low-lying land which can be advantageously filled with sludge and which will serve the purpose for many years to come.

DOSING TANKS—The effluent from the Imhoff tanks will pass over wiers to eight covered reinforced-concrete dosing tanks (one for each filter), 20 ft. square, 6 ft. deep, shaped like an inverted truncated pyramid. Siphons designed to start and stop at predetermined high and low heads will provide automatic, intermittent dosing of the sprinkling filters, thus giving not only short rest periods, but also increasing the uniformity of distribution over the whole surface area.

SPRINKLING FILTERS—Each of the eight sprinkling filters will be 136x319 ft. in plan or about one acre, net, in area, giving an average working rate of 2,125,000 gal. per 24 hours. They will consist of 1½ to 2½-in. broken stone, having an average depth of 6 ft. Concrete floors will be provided, and also "free draining" bottoms for the filters, "consisting of 6-in. half-round pipe, set with convex surface up and laid with open joints."

Each filter will have a 2½-ft. main distributor and 6-in. laterals, all provided with risers which extend to the surface of the filter and are fitted with brass sprinkler nozzles. The automatic dosing tanks will begin discharging when their content is 7½ ft. above the filter surface and stop when it is at 2½ ft.

CHEMICAL HOUSE—On the way to the final settling tanks the filter effluent will pass through a chemical house and receive hypochlorite of lime for disinfection. There will be two dissolving tanks, six solution tanks, and two feed regulators for preparing, standardizing and applying the hypochlorite. To apportion the volume of disinfectant to the volume of flow a 48-in. Venturi meter will be set in the sprinkling filter effluent pipe and properly connected with the feed regulators. The latter will be set to give the proper rate of dosing for a given flow through the meter, and as this flow increases or decreases the difference in pressure in the two pressure chambers of the meter will change the differences in volume of the applied hypochlorite accordingly.

FINAL SETTLING TANKS — The filter effluent, with its dose of hypochlorite, will flow over a set of weirs into final settling tanks, which will serve the double purpose of separating the suspended matter carried forward from the filters and of giving the disinfectant time to act. There will be eight of these tanks. They will follow Imhoff lines, but will be 20⅔ ft. wide, and 100 ft. long, and 11½ ft. deep from the water line to the bottom of the sludge chamber. At the full volume flow of 17,000,000 gal. a day the liquid will be about an hour in passing through these tanks. The sludge space will be small and the sludge will be drawn off frequently. It will flow to a sludge well in the chemical house, from which it will be pumped to and mixed with the crude sewage as the latter enters the screen chamber.

LABORATORY—A fully equipped chemical and bacterial laboratory will be provided at the works.

COST—The tables at the bottom of this column give the costs of the project. The main items of construction cost of the purification works are given in Table I. As already stated, the cost of main and branch sewers, land, etc., brings the total outlay up to $1,080,000. Table II shows operating costs and capital charges.

Acknowledgment is made to E. S. Pierson, of East Orange, chairman of the Joint Committee; to H. T. Stetson, of Orange, and to E. C. Hinck, of Montclair, also to W. D. Willigerod and F. T. Crane, city engineers of East Orange and Orange, respectively.

TABLE I. ESTIMATED CONSTRUCTION COST OF JOINT SEWAGE PURIFICATION WORKS FOR ORANGE, MONTCLAIR AND EAST ORANGE, N. J

(For an estimated population of 133,000 in 1925. Figures do not include sewers, land, etc.)

Screen and overflow chamber	$1,000
Grit chambers	7,000
Main settling tanks	146,000
Sludge drying beds	14,000
Dosing tanks	20,000
Sprinkling filters	280,000
Chemical house and equipment	28,000
Final settling tanks	60,000
Miscellaneous structures	4,000
Miscellaneous piping	18,000
Water supply	6,000
Office and laboratory building	5,000
Laboratory fittings and equipment	3,000
Residence for superintendent	5,000
Miscellaneous work unclassified	3,000
	$600,000
Engineering and contingencies, 15%	90,000
Total estimated cost of construction	$690,000

TABLE II. ESTIMATED ANNUAL OPERATING AND CAPITAL CHARGES, OUTLET SEWERS AND PURIFICATION WORKS FOR ORANGE, MONTCLAIR AND EAST ORANGE, N. J.

(For an estimated population of 133,000)

Operation			
Superintendence and labor:			
Superintendent, 1@$150 per mo.	$1,800		
Chemist, 1@$125 per mo.	1,500		
Laboratory assistant, 1@ $75 per mo	900		
Night attendant, 1@$65 per mo	780		
Laborers, 6@$50 per mo.	3,600	$8,580	
Supplies, power, etc.:			
Hypochlorite of lime, 310 tons@$28.00	$8,680		
Light and heat	640		
Power	720		
Water	1,080		
Laboratory and office supplies	400		
Miscellaneous expenses, repairs and renewals	1,500	13,020	$21,600
Fixed Capital Charges			
Interest 4½% on $1,080,000		$48,600	
Sinking fund, 1.783% on $1,080,000		19,260	67,860
Ttoal estimated annual cost			$89,460

Failure of the Dalton Concrete Core-Wall Dam, Mineville, N. Y.

Dalton dam, a concrete core wall with earth slopes, across an unnamed creek about one mile northwest of Mineville, Essex County, N. Y., gave way on Apr. 23, at 4:25 p.m., and permitted the escape of its empounded reservoir of about 29,000,000 gal. of water down the valley through the towns of Mineville and Moriah Center to a final exit into Lake Champlain, near Port Henry. No lives were lost in the flood and the loss to property was confined to bridges and to auxiliary dams, reservoirs and other equipment of the local mine operators, Witherbee, Sherman & Co., Inc., who are also the owners of the dam that failed. From Guy C. Stoltz, an engineer of the company at Mineville, we have received information from which, with the aid of other data at hand, we have prepared the following account. We are indebted to Mr. Stoltz for the accompanying drawings and views.

The Witherbee-Sherman Co., Inc., owns a number of iron-ore properties in the vicinity of Mineville, and for their development has built and operates several electric power houses, with both steam and water-power prime movers. The Dalton reservoir, which was part of this plant, was not a power reservoir but was used mainly for additional water for cooling air compressors and for the boilers, although eventually it is to be used as a distributing reservoir for the city water supply of Mineville. The dam was built in the fall of 1910 and has successfully held the reservoir since that time.

Fig. 1 shows the reservoir before the failure of the dam and Figs. 3 and 4 the details of the dam itself.

The dam was a concrete core-wall construction founded on the local glacial-drift formation and protected on either side by a sloping earth fill. The core wall was 181 ft. long from the sloping sandy bank on the west side to a wing wall on the east, which wall continued at an angle to the main wall back to a sandy bank on that side. The core wall was 5½ ft. wide at the base and 18 in. wide at the top, with stepped sides at 4-ft. intervals. As shown in the drawings and also in Fig. 6, it was reinforced by longitudinal horizontal 1-in. cables. The ground here is composed entirely of glacial drift, which is generally a stiff sand and gravel mixed with a few large boulders. For the foundation of the higher sections of the dam, where the head reached 20 ft., the excavation was carried down to a depth of 8 ft. below the natural surface in open cribs and the bottom further protected against underflow by tongue-and-groove sheeting driven for a width of some 70 ft. as shown in Fig. 3. No sheeting was driven for the remainder of the section. The drawings call for a length of 6 ft. sheeting, but we are informed that most of the piles were driven to a depth of 8 ft.

A spillway provided as shown in the cuts proved amply sufficient to care for the overflow of water.

The concrete used was a 1:4 mixture of cement and sand, with about 40% additional void-filling large stones, some of which may be noted at the fractured surface in the view in Fig. 6. At the finish of each day's concreting these stones were left with jagged ends projecting to form a bond for the next day's work and when several days would elapse between concreting ⅞-in. iron rods were placed as additional dowel bonds. The resulting concrete is reported to be very good.

The earth fill was made from neighing material. We are not informed re-

FIG. 1. BEFORE

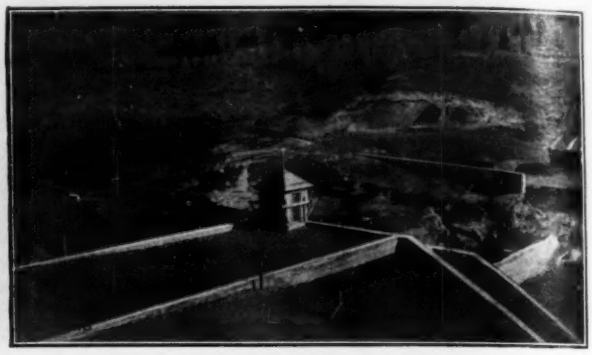


FIG. 2. AFTER

FIGS. 1 AND 2. DALTON DAM BEFORE AND AFTER DESTRUCTION

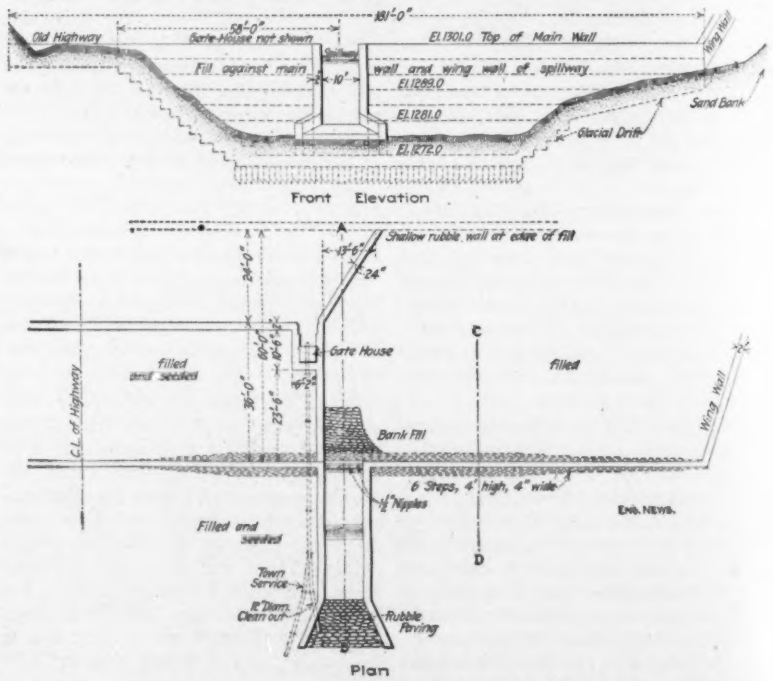


FIG. 3. PLAN AND ELEVATION OF DALTON DAM
(East side at right.)

garding it, except that it was entirely acceptable to the engineers.

The failure occurred after a spell of ordinary spring weather, when the rains were seasonably severe but not exceptional, and when at no time previous had the reservoir overflowed the spillway distance, thus confining the damage to such works as were in or very near those beds.

A highway bridge just below the dam was swept down stream, and at this point a channel 100 ft. wide and 20 ft. deep was cut in the glacial drift. The next upper end of a coal trestle, leading to the central power plant of the company and, together with a mass of trees and timber from the sawmill, improvised a dam which turned the largest mass of water down the road and struck the power plant squarely in the angle be-

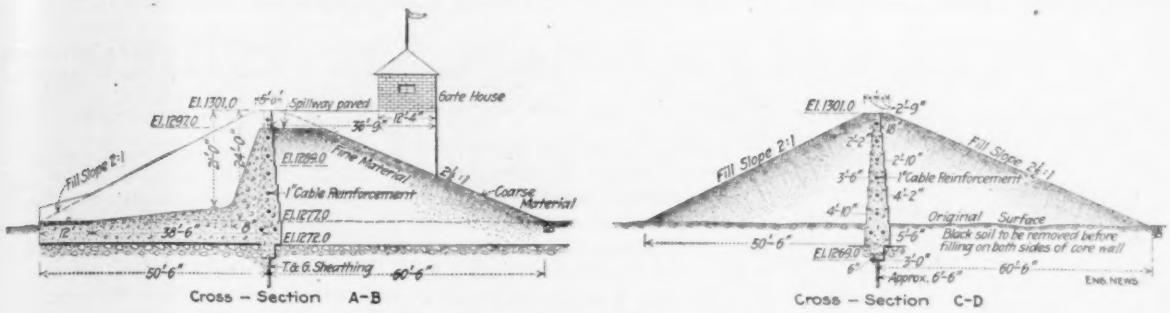


FIG. 4. CROSS-SECTIONS THROUGH DAM

provided. The initial break occurred at the east side near the wing wall, where the prevailing sand bank was opened by the water. This water worked its way through the sand bank and opened a course under the core wall. As soon as an opening was made here, the earth fill on either face of the wall was swept out and the core undermined and pushed out from the far corner of the diagonal wing wall as far along the dam as the east wall of the spillway (Fig. 5). The rest of the dam remains intact. The rushing waters washed out the glacial drift under the dam to a depth of 10 ft., the exposed bottom there being still of the same material. Solid ledge is evidently a considerable depth below the lowest level exposed.

The dam site is about 1300 ft. above and some seven miles away from Lake Champlain, at Port Henry, N. Y. The escaping water, then, had considerable force in its path to the lake, though fortunately that path was in river or creek beds for the greater part of the damage fell to Rowe's pond and dam, an auxiliary reservoir of 1,500,000 gal. capacity. The stone and timber dam was overturned and carried along, cutting out two road crossings.

An overturned barn lodged against the

FIG. 5. VIEW OF DAM AFTER COLLAPSE

tween the main building and an addition containing a recent installation of boilers. The water rose sufficiently high to enter the windows and doors on the main floor. The building diverted the current which swept across the coal storage and carried away about 1000 tons of coal. The brick wall of the boiler house gave away and the flood poured over the boilers and filled the back of the building with sand, boulders and driftwood. The fireboxes were filled with sand and the water rose to the top row of tubes. The fireman and oiler had a narrow escape, but succeeded in getting out on the roof of the transformer house. The dam of the hotpond for boiler and condenser purposes went out. This was a stone dam about 12 ft. in cross-section at the bottom, and some 15 ft. high and 60 ft. wide.

Further along its course the flood tore out another highway bridge, a railway bridge and 125 ft. of embankment and several other yard tracks. The McKinley reservoir dam, a substantial stone and concrete structure, was taken along with the flowage. Below the reservoir a large

FIG. 6. NEAR VIEW OF DESTROYED DAM

air receiver and a portion of a 12-in. air-line pipe leading from the Harmony mines power station to the Old Bed plant was carried away. The old boiler receiver, which would weigh presumably three tons, was carried along for 2000 ft. and deposited on farm land. Another bridge below McKinley pond was swept away and from this point on to Moriah Center, three miles south of Mineville, there is a strip of land averaging 200 ft. wide, which is badly scoured.

A small bridge, together with the large Millbrook bridge at Moriah Center, were taken downstream. One abutment of the Millbrook bridge, 16 ft. high and 25 ft. long, still stands. From Moriah Center to Port Henry on Lake Champlain the damage was only minor, the water being confined practically to the river and creek courses.

Freight Interchange at Chicago

The interchange of through cars from one road to another is one of the complications in handling railway traffic. Chicago is one of the greatest interchange points in the world. Here cars have to be transferred between 20 main-line railways, and the number of cars thus handled is approximately 10,000 per day. As the freight terminal facilities of each line have been developed independently, the terminal system as a whole is complicated and ill arranged, and there are consequently high cost and delay in the transfers.

This article outlines the present facilities and methods, and describes two plans which are proposed in order to reduce the work to a systematic basis and to effect material savings in time, work and cost.

One of the most serious problems in the railway freight-terminal system of Chicago is the handling of interchange freight, as present conditions and facilities are defective as to insuring rapidity, economy and general efficiency. The general situation is that a score of railways enter Chicago and that the interchange business between them is done at a number of individual yards, being supplemented by a complicated system of transfer between the several yards through the medium of belt and connecting lines.

This scattered or decentralized method of handling through or interchange business was described very fully in our issue of Jan. 2, 1902. It was shown then (and the conditions are now much the same, except for increase of traffic) that the 20 trunk lines and the belt and subsidiary lines operated 25 to 30 freight yards, the distance between which ranged from 500 ft. to 16 miles. The round trip between two yards might take as long as five hours, while the transfer of a car from one yard to another might occupy as long as 24 hours. The transfer of cars from yard to yard involved a great amount of empty mileage; thus an engine of one road delivering cars at the yard of another road might return light, leaving the latter road to use an engine of its own in transferring cars to the former road. Much of the work is done, however, by belt and terminal lines, with their own engines.

The complication was so great that it was difficult to secure concerted action or to simplify the transfer arrangements without increasing the delay. The number of interchange trips to each yard was from 1 to 4 per day, and the number of cars interchanged at individual yards was from a very few to as high as 200 or 300. The total number of cars interchanged daily was about 9000 to 10,000.

The description of the interchange system was accompanied by a description of a new interchange yard then recently built in an outlying district (now called Clearing), and designed specially for the handling of interchange business and keeping through cars out of the congested network of lines and yards within the city. The general principle was that through inbound freight trains would run direct to this yard, the cars would be sorted and separated for the various forwarding roads, and either made up into outbound trains for delivery to these roads or sent to their principal outlying yards for this purpose. In this way the local yards and terminals would be relieved entirely of the through-freight business, and the interchange would be done at one point instead of at a number of points. The yard as built formed one of four proposed units, each to be capable of handling 5000 cars daily; its equipment was very complete, the classification being done by gravity switching from a hump track, and the switches of the classification tracks being electrically operated from a tower on a bridge over the summit of the hump.

Unfortunately, this interchange yard has never been used. It was built by an independent company, which found it impossible to get the main-line railways to agree to use it. Its value depended, of course, upon its use by a large number of the roads. It is now owned by the Chicago Union Transfer Ry., and under recent arrangements which are noted below, it seems likely that the facilities of the Clearing interchange yard may soon be utilized for their intended purpose, after having remained idle so long.

Belt Lines and Interchange Yards

The numerous railways converging at Chicago from the north, west and south are connected by three belt lines. Two of these are shown on the accompanying map, together with a proposed new line. The third is too far from the city to handle the class of business under consideration. This is the Elgin, Joliet & Eastern Ry., which extends from Waukegan (on the lake shore), 36 miles north of the city, to West Chicago and Joliet on the west (30 and 37 miles distant), and to Griffith on the south (30 miles distant), with a branch thence northward to South Chicago (see map). The distance from Waukegan to South Chicago by this route is about 130 miles. A belt beyond this is projected by the Chicago, Milwaukee & Gary Ry., and would reach from Milwaukee, Wis., to Gary, Ind.; it is completed from Beloit, Wis., to Aurora, Joliet and Delmar, Ill. (120 miles).

Of the other two existing belt lines, the first or inner belt extends from Cragin south to the Clearing interchange yard (mentioned above), and thence southeast to South Chicago. It lies entirely within the city limits. This is the Belt Line Division of the Chicago & Western Indiana R.R., and includes several freight yards. The second belt line extends from Mayfair west to Franklin Park, south to Argo (on the drainage canal) and Blue Island, and thence east to the lake shore, which it touches at different points between Gary and South Chicago. The distance from Mayfair to South Chicago is about 50 miles. This belt is composed of lines owned by the Baltimore & Ohio R.R. (Chicago Terminal Transfer R.R.) and the Indiana Harbor Belt R.R.

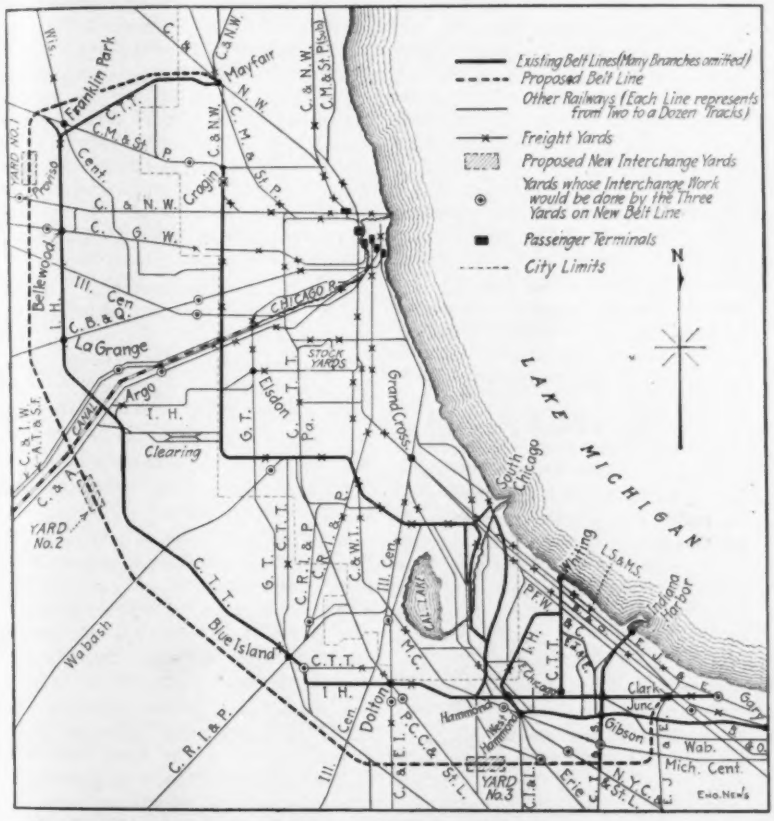
These belt lines are supplemented by numerous branches and spurs connecting with the various railway lines and yards of the general terminal system, and also with various industries and manufacturing plants. It will be seen by the map that both of the belt lines touch the independent interchange yard at Clearing (mentioned above), crossing opposite ends of the property upon which it is located.

Improvements on the Inner Belt System

An important development has been announced recently in regard to the facilities and operation of the inner belt

line, particularly with a view to the improvement of the car-interchange system, and involving the use of the extensive (but hitherto unused) facilities of the Clearing interchange yard. The plan has been agreed upon already by 14 railways. According to this plan, the Belt Line (C. & W. I. R.R.) and the Clearing yard (C. U. T. Ry.) will be leased by a concern representing 14 (and perhaps more) of the trunk lines which enter the city. This concern will operate the system as a complete unit, and in the joint interests of the several railways. With the interchange thus handled by one company, under the coöperation of the trunk lines, the work and cost of car interchange may be made much more systematic, with consequent improvement in speed, economy and general service. It is estimated, also, that by this system of working fully 50% of the interchange cars that now are handled in the various yards in the city, may be kept out of these yards and outside of the congested district of the freight-terminal system.

Improvements will be made in the lines thus acquired, involving an expenditure of several million dollars during the next few years for such work as additional tracks, track elevation, yard extensions, etc. The yard at Clearing will be used for interchange between the east and west and the south and west; while three existing yards on the south line of the belt (near South Chicago) will take care of the interchange between the south and east. It is proposed that eventually a large yard, with capacity for handling 5000 cars daily, would be built at South Chicago to take the place of the three older yards. The Clearing yard now has a capacity for handling 5000 cars per day, but the property provides for three additional units, so that the capacity may be increased ultimately to 20,000 cars per day. This yard would be provided also with transfer facilities for the interchange of freight in less-than-carload (l.c.l.) lots.



SKETCH MAP SHOWING THE CHICAGO RAILWAY TERMINAL SYSTEM, WITH ITS BELT LINE AND FREIGHT YARDS

PROPOSED NEW BELT LINE AND INTERCHANGE YARDS

The plan mentioned above provides only for increasing the capacity of an existing system, but a much broader and more comprehensive scheme has been proposed, which provides for an entirely new set of facilities for handling the complicated and extensive interchange business. This has been worked out by L. C. Fritch, Chief Engineer of the Chicago Great Western Ry., and has been submitted to the railways and the commercial interests. This plan, as shown on the map, includes the construction of a new belt line and three new interchange yards, in place of adapting existing facilities to the needs of the present and the future. The three yards would perform the business now done at the several interchange yards which are indicated by black circles on the map. For some years past Mr. Fritch has had occasion to study the railway terminal problem, both from a traffic point of view and in its relation to the electrification of the entire railway terminal system, which latter idea has been urged at different times on behalf of the city. The interchange situation and the proposed new facilities were presented recently by Mr. Fritch in an address before the Chicago Traffic Club; we are indebted to him for a copy of this address, of which we have made the following abstract as to the new belt line and yards:

A proper system of interchange in Chicago involves the following: 1, a continuous belt railway (see map) extending from a point on Lake Michigan (preferably at or near Clark Junction, Ind.), passing outside of the present belt lines entirely around the Chicago switching district, and terminating on the north near Mayfair, the entire distance being about 50 miles; 2, direct connections with each railway crossed by the main line, with ample sidetrack capacity at each junction point to accommodate 24 hours' interchange traffic with each respective line; 3, a series of three assembling yards, one near Proviso on the north, one opposite the present yard at Clearing, and one near Hammond on the southeast. The estimated cost of such a system of interchange is $15,000,000, and the average cost per car on the basis of 3,000,000 cars interchanged per annum is 66.6c. The cost is distributed as follows: 50 miles of belt line (with multiple tracks) at $125,000 per mile, $6,250,000; yard No. 1, $2,000,000; yard No. 2, $3,500,000; yard No. 3, $2,500,000; equipment, $750,000; total, $15,000,000. The annual charges are estimated at $2,000,000, as follows: interest (5%), $750,000; taxes, $300,000; operation and maintenance, $850,000; incidentals, $100,000.

A few years ago a committee [whose work is mentioned later] ascertained that the actual cost per month for interchange, including yard and transfer service, belt line and trackage charges, but not including interest charges or taxes, or maintenance on property, was $170,939. At this rate the annual charges would be $2,051,268, or an average of $1.12 per car. The total cost per annum, on the basis of 10,000 cars per day, or $3,000,000 per year, would be $3,282,000, as against $2,000,000 under the proposed interchange system. The annual saving under this proposed system would be $1,282,000. The saving effected would be sufficient to retire the funded debt of the company in about ten years.

The cost of the necessary main lines might be reduced if existing outer belt lines were in part purchased for such portions of the distance as would fit into the scheme, but they are not such an essential part of the scheme as would justify their acquisition at a cost greater than that at which they could be reproduced. The existing belt lines would remain valuable as industrial switching lines and would be entirely relieved of interchange traffic, excepting such as would be local to their lines. With interchange traffic provided with an adequate system of transfer, the next step logically would be the acquisition by all the railways in Chicago (under joint ownership and equal trackage rights) of the existing belt lines, in order that all industries in Chicago would be equally accessible to all the roads.

OPERATION—There are three methods of operation applicable to an interchange system: 1, the interchange system to receive and deliver all cars in the home yard of respective railways and perform all work of assembling and distribution of cars in the clearing yards; 2, the respective railways to receive and deliver interchange traffic in one of the clearing yards, and the interchange system to perform all work of assembling and distributing cars in the clearing yards; 3, the respective railways to receive and deliver interchange traffic on receiving and delivery tracks provided at junctions of respective railways with the main line of the interchange system; the interchange system to handle all traffic between clearing yards and these junctions and to perform all work of assembling and distributing cars in the clearing yards. Any one of these methods would be practicable, but it is believed that the interchange system should perform all the work necessary to handle the traffic, relieving the railways of all work incident thereto between home yards and the clearing yards.

The committee recommended an interchange system providing a three-yard scheme, on the theory that it would effect more direct and economical operation than a scheme providing only one clearing yard. It can be seen that a single clearing yard would involve considerable reverse or double movement of cars, which is reduced to a minimum in the three-yard scheme. Furthermore, the entire volume of interchange through one yard would not be as expeditiously nor as economically handled as through a series of yards. There is more possibility for ready expansion of the system with three yards than in a single yard which would ultimately reach such large proportions as to be impracticable.

IMPROVEMENTS IN RAILWAY INTERCHANGE AT CHICAGO

In the address before the Chicago Traffic Club (of which mention has been made) Mr. Fritch reviewed the present conditions and future development of the interchange system, and this part of the address we have abstracted below:

There is as much necessity for a clearing system to interchange the vast traffic among the 30 Chicago railways as there is need for a clearing house for the banks. In no place in the world is such a vast amount of traffic handled and interchanged as in the Chicago railway district. There are approximately 10,000,000 cars (including loaded and empty) forwarded and received in one year by all the railways in the Chicago district. This is equal to about 30,000 cars per day, of which about 33% (or 10,000 cars per day) are interchanged among the various railways in Chicago.

Thus far no comprehensive scheme has been adopted to economically, efficiently and promptly handle this vast interchange traffic. About seven years ago a committee (of which the writer was a member) was engaged on the investigation of this question; it analyzed the operation of each railway and personally inspected every terminal in the Chicago district, occupying about six months in its work. Its report outlined a scheme [described above] which would increase the efficiency and economy of interchanging carload traffic.

The committee estimated that the average time required to effect interchange of a car under present practice was about 72 hours, which could be reduced to 36 hours under the scheme proposed. It found that the cost per month (at that time) for effecting this transfer service was $170,000, including only yard and transfer service, belt and trackage charges; it estimated that under the proposed recommended method the cost would be $115,000 per month, covering not only the operation, but also interest on the cost of the interchange system of yards and belt lines and depreciation thereon.

The method of interchange among the Chicago railways is practically the same as for 40 years past, except that its volume has increased. Anyone familiar with this method is appalled at the enormous waste involved in the way of light transfer-engine mileage, duplication of transfer trips, the lack of conserving time and trackage, and of avoiding congested districts in effecting interchange. The congestion of the inner zone may be relieved by removing this traffic from the inner area, eliminating duplication of traffic and giving up this valuable space to the local traffic which alone belongs in this place. It should be arranged so that not one carload of through traffic should enter the inner zone (with the possible exception of perishable freight).

The economies effected by means of the proposed scheme are largely accounted for in saving of transfer-engine mileage, yard service, eliminating reverse movements, and avoiding belt-line and trackage charges. The committee found that 71 transfer engines were engaged in interchange, while under the proposed scheme 38 engines would perform the service, a saving of nearly 50% in number of engines. Also it required daily 2946 transfer-engine miles, while in the proposed scheme only 1843 miles would be required, a saving of 37%. The number of daily transfer trains was 254 and under the proposed method it would be only 123, a saving of 50%. To the actual economy in cost of service should be added many collateral advantages; such as expediting the movement of traffic and relieving the congestion in inner zone yards. The final results indicate that the railways in Chicago are annually losing sufficient under their present method of interchange to pay for the operation, maintenance, depreciation and fixed charges on a modern system of handling interchange traffic. A summary of the results to be secured from a modern interchange system is as follows:

1. Reducing the average time in effecting deliveries of cars to connecting lines by about 50%.

2. Relieving the existing yards to the extent of making them available for local traffic and (in many cases) obviating the necessity of increasing yard facilities.

3. Relieving the congestion of main lines in the inner zone, thus expediting the movement of local traffic.

4. Reducing the cost of yard operation in existing yards where it is not necessary to assemble cars for each connecting line, whereas solid trains not assembled in road order would be delivered to the interchange yard where the necessary assembling would be done.

5. Reduction in cost of actual transfer service now requiring individual trains to each connecting line (with often light trains), as against solid maximum trains containing cars for various lines to be delivered to the interchange system.

6. The possibility of running solid road trains into and out of the interchange yards without entering existing yards. Thus at the first freight terminal nearest Chicago, a road could separate through from local traffic, the local traffic going to the home yard and the through traffic to the interchange yard direct, thereby saving in time and expense of rehandling in home yards. On outbound traffic, solid trains could be taken from interchange yards by various roads without handling them through home yards in Chicago.

GENERAL TRAFFIC CONDITIONS

There are 50 railway companies in the Chicago district, including the belt lines and subsidiary corporations belonging to various trunk lines. Eliminating the subsidiary lines, there are 30 railways which

would be directly affected by an interchange system and would necessarily become users of such a system. There are 23 trunk lines (or railway systems) entering Chicago, representing about 33% of the entire mileage of the United States. There is more railway traffic handled within the area of the Chicago railway district than in any equal area in the world. The trend of railway traffic passing through Chicago is east and west, and classified in this way there are 10 western trunk lines and 13 eastern lines. Of the total forwarded and received business, the western lines handle about 66% and the eastern 24%. Of the total inbound business, the western lines handle 66% and eastern lines 34%. Of the outbound business, they handle 67% and 33%, respectively. Nearly 30% of all cars forwarded and received by Chicago railways are in through traffic, originating beyond and destined to points beyond Chicago. It is this particular traffic which will be best served by an adequate interchange system.

The problem of adequately providing for the handling of the vast terminal traffic of Chicago railways will require an early solution. The longer it is deferred, the more complex and difficult of solution it will become. While the above remarks deal particularly with through-freight traffic, the proper handling of local-freight traffic and the passenger traffic are important elements of the problem. The solution with respect to the handling of the interchange traffic is a distinct and separate element, but it is of such importance that it should be solved with the freight-traffic problem. The area devoted to railway terminals in the heart of the city is rapidly encroaching upon and seriously affecting the expansion of the business district, and the time will soon come (if it is not already reached) when the value of land in these inner areas will make it prohibitive to put it to uses such as interchanging carload traffic, one car occupying a space valued at from $18,000 to $20,000. The tendency for some years past has been to locate large freight yards from 15 to 20 miles from the business district, and with this end in view the belt and three-yard interchange system here proposed has been suggested.

Notes from Engineering Schools

PRINCETON UNIVERSITY—On Apr. 13, at the New York Princeton Club, the Princeton Engineering Association was organized with 100 members. The object of the society is to advance the interest and influence of the Engineering Department of the University and to promote the welfare of its alumni. W. P. Field, of Newark, N. J., was elected President; Wilbur Fisk, of New York City, Vice-President, and C. H. Higgins, of Jersey City, N. J., Secretary and Treasurer.

MICHIGAN COLLEGE OF MINES—A course in technical journalism and English is to be given to first-, second- and third-year students. This work will be compulsory during the first two years, and in the junior year two elective courses will be offered—one in technical journalism and one in business correspondence. The present course was preceded by a course in technical writing offered experimentally to upper classmen, which brought to light the need for elementary instruction in writing English during the first two years.

The American Steamer "Texas" was sunk near the mouth of the Gulf of Smyrna, Apr. 29, by an explosion, and 69 of her 139 passengers lost their lives. One report states that the vessel was sunk by a Turkish mine after being warned of her danger by blank shots fired from Yenikale Fort. Another has it that a shot from the fort caused the explosion and that the captain of the ship is held as a spy by the Turkish government. The "Texas" was a vessel of 480 tons, owned by the Archipelago-American Steamship Co. She was transferred from the Turkish to the American flag in 1910.

A Derailment at Low Speed on the New Orleans & Northeastern R.R., near Hattiesburg, Miss., May 6, resulted in the death of nine persons. A Confederate veteran's excursion special left the track and the locomotive, baggage car, one day coach and three tourist sleepers rolled down an embankment. Reports state that the derailment occurred on a tangent at a speed of 30 miles per hour.

A Powder Mill Explosion at Hazardsville, Conn., May 6, badly damaged the plant of the Du Pont powder company and shook the surrounding country, according to press reports, for a distance of 25 miles. No one was killed, however.

A Japanese Coal Mine Explosion is reported in a press dispatch of Apr. 30, at the mine of the Hokkaido Co., in Yubari, on the Island of Yezo. The report states that the colliery was wrecked and that 283 miners were entombed with little hope of rescue.

A Conflagration in Damascus, Apr. 26-27, is reported to have destroyed the "bazaar quarters," comprising miles of long streets covered in with wooden canopies and lined with shops and warehouses. The loss is estimated at $10,000,000.

The Mississippi River Floods continued to be most dangerous in the lower river during the past week. Rains all over the Mississippi valley tended to increase the stages from Cairo south. At Cairo the falling waters were checked and a rise of 3 ft. over the past week recorded. Similar conditions prevailed all the way down the river, but with no dangerous results until Louisiana was reached. All the river counties in the State of Louisiana are in an extreme condition of flood. The area of land under water is difficult of determination, and the number of refugees equally so, but it is known that conditions are worse than ever before in the history of the river. The following table shows the stages of the river as reported by the U. S. Weather Bureau on each of the three preceding Tuesdays:

	Apr. 23	Apr. 30	May 7*
Cairo	44.8	45.5	48.8 f.
Memphis	39.9	36.8	38.0 r.
Helena	54.2	51.3	48.5 s.
Arkansas City	52.9	52.1	51.5 s.
Greenville	46.8	45.6	44.9 f.
Vicksburg	48.4	47.9	48.4
Natchez	50.1	49.7	50.8 r.
Donaldsonville	32.2	...	34.3 r.
Baton Rouge	....	41.5	43.2 r.
New Orleans	20.2	20.4	21.2 r.

*f = falling.
r = rising.
s = stationary.

It will be noted from this table that the upper river is now somewhat higher than it was two weeks ago, and that the lower river is considerably higher. The danger stage at New Orleans is 18 ft. The U. S. Weather Bureau predicts a stage of 21.5 at New Orleans, 51.3 at Natchez and 43.9 at Baton Rouge, on May 12. The greatest danger point along the river at present is at New Orleans. On May 1 the Torras levee, some 60 miles north of Baton Rouge, gave way and flooded the entire surrounding country. On May 7 this crevasse had reached a width of 2000 ft. and was washing fast, although the Army engineers were devoting their best efforts to check its growth. The pressure of the flood was relieved by this break, however, and it is expected that the New Orleans stage will be much reduced thereby. Federal and state aid is being extended to the thousands of refugees along the course of the flood, but great difficulty is being experienced in obtaining sufficient labor to build the necessary protective works on the river just above New Orleans.

A Rear Collision at Odessa, Minn., on the Chicago, Milwaukee & St. Paul Ry., occurred on Dec. 28, 1911, and caused the death of 10 persons. A report of the investigation of this accident has just been issued by the Interstate Commerce Commission. While there is a block system in use on this division of the railway, the investigation indicated that the rules governing the use of the block system were not properly understood by the employees, nor was obedience to the rules enforced by the operating officials. It appears from the investigation that a station operator let a second train into the block and gave it a clearance card, although he knew that the train ahead had not yet cleared the block.

A notable feature of the accident was that the cars in which the fatalities occurred were of steel construction. The locomotive of the rear train pushed its way several feet into the rear end of a sleeping car, which was the rear car of the standing train, and caused the rear end of the next car ahead, a dining car, to telescope the sleeping-car for nearly half its length. According to the report, the locomotive of the rear train, in colliding with the rear end of the sleeping-car, raised it somewhat and depressed its front end, permitting the underframe at the front end of the sleeping-car to slide under the underframe of the dining-car platform. Thus the solid steel underframe of the dining-car sheared off the superstructure of the sleeping-car, as the two were forced together. The underframes of both cars were little damaged, and the running gear was so little injured that both cars were hauled nearly 500 miles to the shops on their own wheels. The accident is, therefore,

an illustration of the need of careful design of steel-car ends to hold the underframes in line in the event of collision.

Approaching Completion of the Gatun Dam—The inner sides of the north and south toes of the east wing of Gatun Dam have been brought together for a distance of about a hundred yards from the spillway toward the locks, and this portion of Gatun Dam is practically completed. Over the hydraulic fill, the top of which was at elevation 93 above sea level and about 200 ft. wide at the spillway end, clay has been dumped and puddled between the rock toes until the top of the dam is practically level for a width of 100 ft. at present elevation of 103.35 ft. Settling will decrease this slightly, but it is probable that a coating of rock will be laid to make the final top 105 ft. above sea level.

Filling in that portion of the east wing in which the toes have not come together, over a distance of about 600 yd., is continued by means of clay from the borrow pit at the west end of the dam. On the west wing, hydraulic filling is still under way and has reachd an elevation of about 83 ft.—"Canal Record," Apr. 3.

One of the Panama Canal Lock Gate leaves has been completed, and on Apr. 15 was hung in its final position in the upper east chamber of the Gatun locks, practically ending the contractor's work in connection with this leaf. The operation of hanging the leaf consisted of moving it horizontally 4 ft. and lowering it about 8 in. from the foundation on which it was erected, until the hemispherical cavity in the heel of the leaf fitted over the solid hemisphere of nickel steel, 16 in. in diameter, which forms the bottom bearing or hinge. At the top the hinge consists of a 10-in. pin, engaged in a yoke casting. Each of these upper guard gate leaves is 54 ft. 8 in. high and weighs approximately 400 tons.

When hung the leaves are swung back against the wall into recesses so made that when the gate is open the outer sides of the leaves are 1 ft. back of the faces of the lock walls, between which there is a clearance of 110 ft.

The installation and adjustment of the machinery for opening and closing the gates is to be done by the First Division of the Chief Engineer's Office. The first set of machinery for the operation of the gates was completed on Jan. 13, except the strut connection, which is not installed until the gate is completed. The second set of machinery has been shipped and will soon be installed and tested on the other leaf of the upper guard gate.

The builders of the lock gates, the McClintic-Marshall Co., of Pittsburg, Penn., have now a force of about 1000 men on the Isthmus.

The Lotschberg Tunnel, which pierces the mountains on the right bank of the Rhone River to form a northward outlet from the Simplon Tunnel, is near completion. In March, 1912, the full-section excavation was completed, and only about 500 ft. of lining remained to be placed. It was expected that the lining would be finished in the latter half of April. The tunnel is double-track, is 9.03 miles (47,676 ft.) long, and is lined for its full length.

Grade Crossing Elimination Work in New York City has been completely checked, according to Chairman Willcox, of the Public Service Commission, by the action of Governor Dix in vetoing the appropriation of $350,000 granted by the Legislature for this work. The reason given by the Governor for his veto was the fact that there remains a considerable balance from former appropriations. Mr. Willcox, in an official protest entered Apr. 23, explained the apparent surplus as follows:

Under the law, a quarter of the cost is paid by the state, a quarter by the city and the rest by the railroad company. Another provision of the law is that no payment can be made to any railroad company on account of work done until that work is completed. It is for this reason that practically the whole of the $500,000 already appropriated for the grade-crossing work of this commission remains on the state books as unexpended. It is to this, apparently, that the Governor refers when he speaks of "large balances available for these purposes." It is now impossible for the city to go ahead, as the grade-crossing act provides that no determination of any case can be made unless there has been an appropriation by the State Legislature.

Chairman Willcox states in his protest that there are more than 400 grade crossings in New York City, at which 90 persons have been killed and 136 injured during the past four years. If the state appropriation of $350,000 had been allowed to stand, it would have resulted in the undertaking of $1,400,000 worth of work and would have resulted in the elimination of 15 crossings on the Long Island R.R. and five on the Staten Island Ry.

The American Museum of Safety has announced the receipt of a gift of $5000 presented by Judge Elbert H. Gary on behalf of the U. S. Steel Corporation. The money is to be used toward obtaining a collection of the best devices for saving life at sea, a. a permanent public exhibit for demonstration and study. Another gift of $500^ has been presented anonymously for research work in connection with industrial poisons.

Mr. Arthur Williams has recently become president of the Museum, succeeding Philip T. Dodge, who was obliged to withdraw on account of ill health. The Museum, at 29 West 39th St., New York, is open to the public between 9 a.m. and 5 p.m. every day. Anyone with practical ideas for life saving at sea is requested to submit them at the Museum.

A Prize of $4000 is to be awarded in 1914 for the best original paper upon the scientific advance and progress in the |technical application of electricity presented to the "Association des Ingenieurs Electriciens Sortis de l'Institut Electrotechnique Montefiore," which is located at 31 Rue Saint Gilles, Liège, Belgium. This prize carries the name "Fondation George Montefiore Levi," and is the triennial income of 150,000 francs bequest made by the late George Montefiore. The paper must be presented at the above address before Mar. 31, 1914, must be written in either English or French and should be either typewritten or printed. The precise nature required of the articles is not stated in the program of the prizes, which has been sent out from Liège, but it is stated there that the papers should be anonymous and should have certain uniformities which can be ascertained by writing the home office.

Good Roads and Matrimony were set down as cause and effect in an address, reported by the Philadelphia "Public Ledger," to have been given by Bishop Samuel Fallows, of Illinois, in a recent address before the Illinois Woman's State Good Roads Convention:

Of course the man who is desperately in love will not be kept from the object of his affections by poor roads, but in the rural sections there are many estimable young men of faint heart who might be deterred in their courting by muddy roads. Good roads are necessary first to make courtship easy. After courtship comes the home, and the home supports the Church.

Personals

Mr. J. R. Ellis, City Engineer of Columbia, Mo., has resigned to become Assistant City Engineer of Regina, Sask., Canada.

Mr. Horace McEwan has been appointed Superintendent of the Prince Edward Island Ry., of Canada, succeeding Mr. G. A. Sharp, resigned.

Mr. James E. Carroll, for nine years past City Engineer of Crookston, Minn., has been appointed Assistant Commissioner of Public Works, of St. Paul, Minn.

Dr. Arthur B. Lamb, Professor of chemistry at New York University, New York City, has been appointed Assistant Professor of chemistry at Harvard University.

Dr. Rudolph Diesel, inventor of the Diesel engine, was elected an honorary member of the American Society of Mechanical Engineers at the meeting held on Apr. 30.

Mr. E. S. Stuart has resigned from the Bureau of Explosives of the Interstate Commerce Commission to become Special Agent of the Buffalo, Rochester & Pittsburgh R.R.

Mr. Harry A. Hey, Jun. Am. Soc. M. E., Office Manager of the American Society of Mechanical Engineers, was married on May 1, to Miss Caroline Lohmann, of Wilkes-Barre, Penn.

Mr. C. O. Davis, formerly Roadmaster of the Milwaukee Electric Ry. & Light Co., has been appointed Superintendent of street cleaning and garbage collection, of Milwaukee, Wis.

Mr. James Dugald White, son of Mr. J. G. White, M. Am. Inst. E. E., was married at Montclair, N. J., on May 4, to Miss Dorothy Owen, daughter of Mr. James Owen, M. Am. Soc. C. E.

Mr. J. G. Taylor, General Superintendent of the Lake Superior division of the Canadian Pacific Ry., at North Bay, Ont., has been transferred to the Saskatchewan division, at Moose Jaw, Sask., succeeding Mr. J. J. Scully, who goes to North Bay.

Mr. Mark A. Replogle, M. Am. Soc. M. E., Sales Engineer of the Lombard Governor Co., Ashland, Mass., has been appointed District Agent for the Northern Mississippi River and Lake States sections of the company, with headquarters in the Fisher Bldg., Chicago, Ill.

Mr. Edward C. Sherman, M. Am. Soc. C. E., whose resignation as Designing Engineer with the Isthmian Canal Commission was noted in our columns on May 18, 1911, has opened an office at 6 Beacon St., Boston, Mass., for the general practice of civil engineering.

Mr. Theodore E. Videto, for nearly seven years Architect of the Department of Parks, Boroughs of Manhattan and Richmond, New York City, has resigned and will become Assistant Superintendent of building construction of the Canadian Pacific Ry., at Montreal Que.

Mr. Edward G. Connette, Assoc. Am. Inst. E. E., whose resignation as Transportation Engineer of the Public Ser-

vice Commission, First District, New York, was noted in our issue of Apr. 18, has been made Vice-President of the International Traction Co., of Buffalo, N. Y.

Mr. Roy H. Pinkley, Superintendent of Buildings of the Milwaukee Electric Ry. & Light Co., Milwaukee, Wis., has been appointed Superintendent of construction and maintenance-of-way, succeeding Mr. F. G. Simmons, whose appointment as Commissioner of Public Works was noted last week.

Mr. George C. Jones, General Manager of the Central Vermont Ry., has been appointed Vice-President in charge of construction and operation of the Southern New England Ry., in addition to his other duties. Both lines are subsidiaries of the Grand Trunk Ry.

Mr. H. C. Estep, formerly Division Engineer on the construction of the Chicago, Milwaukee & Puget Sound Ry., and recently in charge of location surveys for the Southern New England Ry., has been appointed Engineer of Construction of the latter, which is a subsidiary of the Grand Trunk Ry. of Canada.

Mr. George V. Rhines, M. Am. Soc. C. E., formerly Structural Engineer with George S. Mills, Architect, Toledo, Ohio, has entered a partnership with Messrs. George S. Mills, Lawrence S. Bellman and Charles M. Nordhoff, under the firm name of Mills, Rhines, Bellman & Nordhoff, Architects, with offices at 1234 Ohio Bldg., Toledo, Ohio.

Prof. John H. Nelson, of the department of applied mechanics, Case School of Applied Science, Cleveland, Ohio, has been appointed Professor of applied mechanics at the Worcester Polytechnic Institute, Worcester, Mass., succeeding the late Edward L. Hancock. Prof. Nelson is a graduate of South Dakota State College and of the University of Illinois.

Mr. John L. Weller, M. Can. Soc. C. E., has been appointed Engineer in charge of the design and construction of the new Welland Ship Canal, a waterway 30 ft. deep and 200 ft. wide to be built between Lakes Erie and Ontario in Canada. Mr. Weller has been Superintending Engineer of the present Welland Canal, at St. Catharines, Ont., for a number of years.

Mr. E. J. Wright, Division Superintendent of the New York Central & Hudson River R.R., at New York City, has been appointed General Superintendent of the Eastern district, with headquarters at Albany, N. Y., succeeding Mr. W. J. Fripp, who was recently promoted to be Assistant General Manager. Mr. C. Christie, formerly Superintendent of the St. Lawrence division of the Rome, Watertown and Ogdensburg R.R., succeeds Mr. Wright at New York City.

Mr. Henry E. Riggs, M. Am. Soc. C. E., of the Riggs & Sherman Co., Consulting Engineers, of Toledo, Ohio, has been appointed Professor and head of the department of civil engineering at the University of Michigan. Prof. Riggs graduated from the University of Kansas in 1886. He spent a number of years in railway work and was for six years Chief Engineer of the Toledo, Ann Arbor & North Michigan Ry. Since 1896 he has been in private practice in Toledo.

Messrs. W. P. Anderson, Past President, Can. Soc. C. E., Chief Engineer of the Department of Marine and Fisheries of Ottawa; John Kennedy, Hon. M. Can. Soc. C. E., Consulting Engineer for the Harbor Commissioners of Montreal; J. S. Armstrong, M. Can. Soc. C. E., Consulting Engineer, of St. John, N. B., and F. W. Cowie, M. Can. Soc. C. E., Chief Engineer for the Montreal Harbor Commissioners, are to be the Canadian delegates to the International Navigation Congress, at Philadelphia, Penn., May 23-28.

Mr. George L. R. French, Division Superintendent of the Boston & Maine R.R., at Boston, Mass., has been appointed General Superintendent of the Rutland R.R., at Rutland, Vt. Mr. French graduated in civil engineering from the Massachusetts Institute of Technology in 1884, and the following year entered the service of the Chicago, Burlington & Quincy R.R., as a rodman. In 1890 he returned to the East to become an assistant engineer with the Boston & Maine R.R. Since 1907 he has been Superintendent of the terminal division at Boston.

Mr. Frank W. Hodgdon, M. Am. Soc. C. E., Chief Engineer of the Massachusetts Board of Harbor and Land Commissioners, has been appointed Chief Engineer of the Port of Boston, Mass. The development and administration of the Port of Boston was recently placed in the hands of a Board of Directors, of which Gen. Hugh Bancroft, of Boston, is Chairman. Mr. Hodgdon is a graduate of the Massachusetts Institute of Technology, class of 1876, and has been Chief Engineer of the Massachusetts Board of Harbor and Land Commissioners for a number of years.

Mr. Richard C. St. John, formerly Assistant Engineer of the Canadian Pacific Ry., at Winnipeg, Man., has been promoted to be Engineer of Maintenance-of-Way of the Western lines, with headquarters at Winnipeg. Mr. St. John is a Canadian by birth and received his engineering education at Cornell University, where he graduated in 1887. Previous to his work with the Canadian Pacific Ry., he had been Assistant Chief Engineer of the Michigan Central R.R., Assistant Superintendent of the Great Northern Ry., Principal Assistant Engineer of the Missouri Pacific Ry., and Assistant Subway Engineer of Chicago, Ill.

Mr. Horace W. King, for the past few years with the Arnold Co., of Chicago, Ill., in charge of hydro-electric and irrigation developments in various parts of the country, has been appointed Professor of hydraulics and irrigation engineering at the University of Michigan. Prof. King is a graduate of the University of Michigan, class of 1895, and has had a wide experience in engineering work, including surveys for the Nicaragua Canal, in the Philippine Islands, on the Canton-Hankow Ry. in China and with the United States Reclamation Service. He was the engineer in charge of the construction of the Umatilla project in Oregon.

Mr. W. D. Weaver, M. Am. Inst. E. E., has retired from active work as Editor of the "Electrical World," although he will continue to devote some time as Consulting Editor. Mr. Weaver began editorial work in 1893 as Editor of the original "Electrical World"; later he was Editor of the "American Electrician," which he helped to found. Upon the consolidation of the "Electrical World" and the "Electrical Engineer," Mr. Weaver and Mr. T. C. Martin became Editors of the combined journal, which is the present "Electrical World." Mr. Weaver was educated at the United States Naval Academy, at Annapolis, Md., where he graduated in 1880. For the 12 years following he served in the U. S. Navy an an engineer officer, and again during the Spanish-American War as a volunteer. For a short time before entering upon his editorial work he was engaged in the manufacture of electrical instruments.

Obituary

Donald M. Scott, City Engineer of Laurium, Mich., committed suicide on Apr. 29. He was 55 years old.

J. P. Mabee, Chairman of the Canadian Railway Commission, died on May 6, following an operation for appendicitis. Previous to becoming Chairman of the Railway Commission he was Judge of the High Court of Justice of Ontario. He had also served as a Canadian representative on the International Waterways Commission.

Hinsdill Parsons, Vice-President and General Counsel of the General Electric Co., of Schenectady, N. Y., was killed in an automobile accident near East Greenbush, N. Y., on Apr. 28. He was 48 years old and was one of the most prominent corporation lawyers of the country. He had been associated with the General Electric Co. since 1894.

Stephen Samuel Haight, M. Am. Soc. C. E., a civil engineer and city surveyor, of New York City, died at St. Luke's Hospital, New York, on May 1. He was born in New York City in 1837. His engineering work began in 1865, when he entered the employ of Thomas C. Cornell, a civil engineer of Chappaqua, N. Y., with whom he later became a partner. Some years later he came to New York City and for a number of years was an assistant engineer in the Department of Parks, during which period he supervised the street layouts in various parts of the Borough of the Bronx. He leaves a widow and two children.

James Oborne, General Superintendent of the British Columbia division of the Canadian Pacific Ry., at Vancouver, B. C., died on May 1. He was born in Montreal, Que., Sept. 19, 1861, and after a common school education entered the service of the Grand Trunk Ry., as a clerk. After nine years in clerical positions with the Grand Trunk Ry., he became chief clerk to the mechanical superintendent of the Canadian Pacific Ry. In 1890 he was appointed Superintendent of Car Service, and in 1896 Assistant to the Vice-President. Since 1899 he had served as General Superintendent of nearly every division of the Canadian Pacific Ry.

James Rawle, President of the J. G. Brill Co., of Philadelphia, Penn., died at his home in Bryn Mawr, on Apr. 30. He was one of the founders of this great street-car building enterprise. Mr. Rawle was born at Lancaster, Penn., Nov. 15, 1842. He graduated from the University of Pennsylvania in 1861. From 1862 to 1872 he was employed in civil engineering work on the Philadelphia & Erie R.R., and was a contemporary of the late A. J. Cassatt, President of the Pennsylvania R.R., who was then a resident engineer on the Philadelphia & Erie. In 1872 Mr. Rawle became a partner of the late John G. Brill and George M. Brill, under the firm name of J. G. Brill & Co. The firm was incorporated in 1887 as the J. G. Brill Co., of which Mr. Rawle was Secretary and Treasurer. Upon the death of G. M. Brill, in 1906, he succeeded to the office of President, which position he had since held.

Engineering Societies

COMING MEETINGS

NATIONAL FIRE PROTECTION ASSOCIATION.
May 14-16. Annual meeting at Chicago. Secy., F. H. Wentworth, 87 Milk St., Boston, Mass.

MASTER BOILER MAKERS ASSOCIATION.
May 14-17. Annual convention at Pittsburg, Penn. Secy., H. D. Vought, 95 Liberty St., New York.

AMERICAN RAILWAY ASSOCIATION.
May 15. Semi-annual meeting at New York City. Secy., W. F. Allen, 75 Church St., New York.

OHIO SOCIETY OF MECHANICAL, ELECTRICAL AND STEAM ENGINEERS.
May 16-18. Annual meeting at Pittsburg, Penn. Secy., F. E. Sanborn, Ohio State University, Columbus, Ohio.

INTERNATIONAL RAILWAY FUEL ASSOCIATION.
May 22-25. Annual convention at Chicago, Ill. Secy., D. B. Sebastian, La Salle St. station, Chicago.

INTERNATIONAL NAVIGATION CONGRESS.
May 23-28. At Philadelphia, Penn. Secy., J. C. Sanford, 344 The Bourse, Philadelphia, Penn.

AMERICAN SOCIETY OF MECHANICAL ENGINEERS.
May 28-31. Spring meeting at Cleveland, Ohio. Secy., C. W. Rice, 29 West 39th St., New York.

AMERICAN WATER WORKS ASSOCIATION.
June 3-8. Annual convention at Louisville, Ky. Secy., J. M. Diven, 271 River St., Troy, N. Y.

ASSOCIATION OF RAILWAY TELEGRAPH SUPERINTENDENTS.
June 4. Annual convention at New York City. Secy., P. W. Drew, 800 Line, Chicago, Ill.

NATIONAL ELECTRIC LIGHT ASSOCIATION.
June 10-14. Annual meeting at Seattle, Wash. Secy., T. C. Martin, 29 West 39th St., New York.

MASTER CAR BUILDERS' ASSOCIATION.
June 12-14. Annual convention at Atlantic City, N. J. Secy., Jos. W. Taylor, 390 Old Colony Bldg., Chicago.

AMERICAN RAILWAY MASTER MECHANICS' ASSOCIATION.
June 17-19. Annual convention at Atlantic City, N. J. Secy., Jos. W. Taylor, 390 Old Colony Bldg., Chicago.

CANADIAN ELECTRICAL ASSOCIATION.
June 19-21. Annual meeting at Ottawa, Ont. Secy., T. S. Young, 220 King St., West, Toronto, Ont.

AMERICAN INSTITUTE OF ELECTRICAL ENGINEERS.
June 25-28. Annual convention at Boston, Mass. Secy., F. L. Hutchison, 33 West 39th St., New York.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
June 25-28. Annual convention at Seattle, Wash. Secy., C. W. Hunt, 220 West 57th St., New York.

SOCIETY FOR THE PROMOTION OF ENGINEERING EDUCATION.
June 26-28. Annual meeting at Boston, Mass. Secy., H. H. Norris, Cornell University, Ithaca, N. Y.

SOCIETY OF AUTOMOBILE ENGINEERS.
June 27-29. Summer meeting at Detroit, Mich. Secy., C. F. Clarkson, 1451 Broadway, New York.

NATIONAL MUNICIPAL LEAGUE.
July 8-12. Annual meeting at Los Angeles, Calif. Secy., C. R. Woodruff, North American Bldg., Philadelphia, Penn.

National Supply and Machinery Dealers' Association—A triple joint convention is to be held in coöperation with the Southern Supply and Machinery Dealers' Association and the American Supply and Machinery Manufacturers' Association at Norfolk, Va., May 13-15.

Iowa District Gas Association—The eighth annual meeting will be held in Lincoln, Neb., May 22 to 24, with headquarters at the Lincoln Hotel. The following papers will be presented: "The Prepayment Meter—Its Use and Abuse," E. C. Weisgerber, Cedar Rapids, Ia.; "Does the Consumer's Equipment Inspection Increase Sales of Gas to Existing Consumers?" S. E. Linton, Jr., Sioux Falls, S. D.; "Construction and Operation of a Modern Water Gas Works for a Small Town," R. K. Runner, Charles City, Ia.; "Surface Combustion," H. C. Blackwell, Davenport, Ia.; "Availability of Ammonia Machines for Small Plants," C. W. Lockwood, Clinton, Ia.; "Methods of Increasing the Efficiency of Employees," J. A. Perry, Omaha, Neb. The secretary is G. I. Vincent, Des Moines, Ia.

National Fire Protection Association—The program for the 16th annual meeting to be held in Chicago, Ill., May 14-16, includes the following papers and committee reports: May 14—"High Pressure Fire Service Systems," H. B. Machen, New York (Chairman); "Caliber of Fire Streams," Charles H. Fox, Cincinnati. May 15—"Cold Storage Warehouses," E. P. Boone, New York (Chairman); "Automatic Garages," F. E. Cabot, Boston (Chairman); "Electric Railway, Light and Power Properties," C. H. Patton, Cleveland (Chairman); "Automatic Sprinklers," E. P. Boone, New York (Chairman). May 16—"Fireproof Construction, Including Concrete and Reinforced Concrete," Ira H. Woolson, New York (Chairman); "Fire Protection Coverings for Window and Door Openings," W. C. Robinson, Chicago (Chairman); "Gravity Tanks," Gorham Dana, Boston (Chairman).

International Railway Fuel Association—The program of papers for the fourth annual convention at Chicago, Ill., May 22-25, is as follows: "Standard Locomotive Fuel Performance Sheet," R. Collett; "Fuel as a factor in Locomotive Capacity," W. F. M. Goss; "The Use of Anthracite Coal for Locomotive Fuel," T. S. Lloyd; "Proper Method of Firing Locomotives," D. C. Buell; "Locomotive Drafting and Its Relation to Fuel Consumption," H. B. MacFarland; "Inspection of Fuel From the Standpoint of the Producer and Consumers," J. E. Hitt and Glenn Warner.

The headquarters for the convention will be at the Hotel Sherman. There will be an exhibition of mining machinery and coal-handling devices in a room adjoining the one reserved for the convention. The sessions will be called to order at 9 a.m. each day. The first two papers named in the list above will be presented at the Wednesday session, the next two at the Thursday session, the fifth at the Friday session and the last at the Saturday session.

International Association for Testing Materials—The sixth congress will be held in the Engineering Societies' Bldg., in New York City, during the first week of September. On Monday of that week, Sept. 2, the headquarters will be open for registration of members, and in the evening there will be an informal reception under the joint auspices of the American Society For Testing Materials, the American Institute of Electrical Engineers, the American Society of Mechanical Engineers and the American Institute of Mining Engineers. On Tuesday morning following the address of welcome by Dr. Henry M. Howe and other addresses, the various sections of the congress will arrange for their individual sessions. Sessions will be held on Tuesday afternoon, Wednesday morning and in both the morning and afternoon on Thursday and Friday. It is announced that more than 150 reports and papers are promised, of which number about one-half have already been received. About 35 American papers are included in the list. The proceedings of the congress will be conducted in English, German and French, with the aid of interpreters.

The entertainment features, in addition to the informal reception on Monday evening, will include an excursion to West Point on Wednesday afternoon and evening, a reception tendered by the American Society of Civil Engineers on Thursday evening, and an excursion in New York harbor on Friday evening. Following the adjournment of the congress at noon on Sept. 7 there will be an excursion by special train to Washington, Pittsburg, Buffalo and the Lehigh Valley at a cost of $50, for the benefit of those who wish to see something of the country and its industrial development.

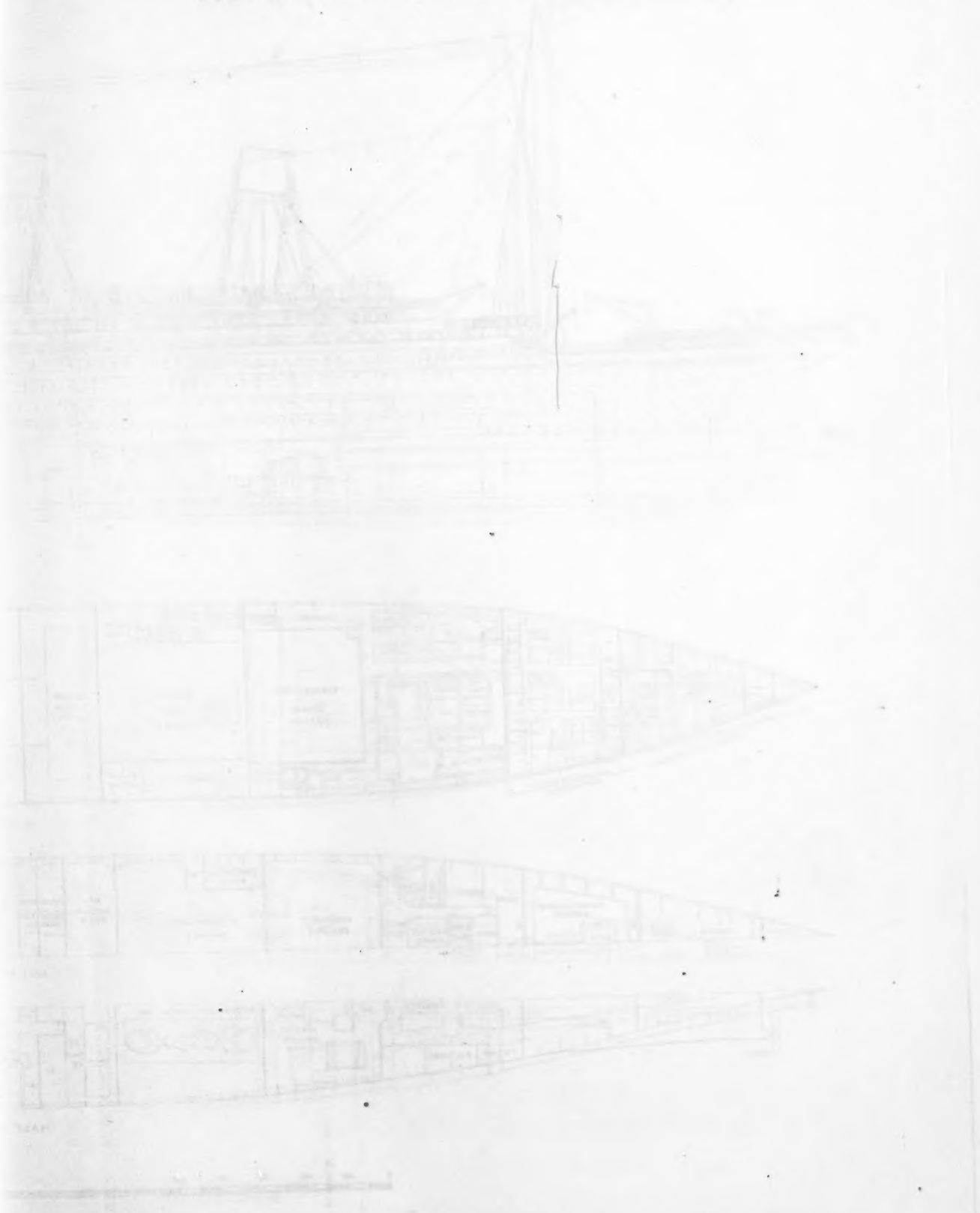
The congress fee is $5 for members and delegates and $3 for ladies and guests not taking part in the deliberations.

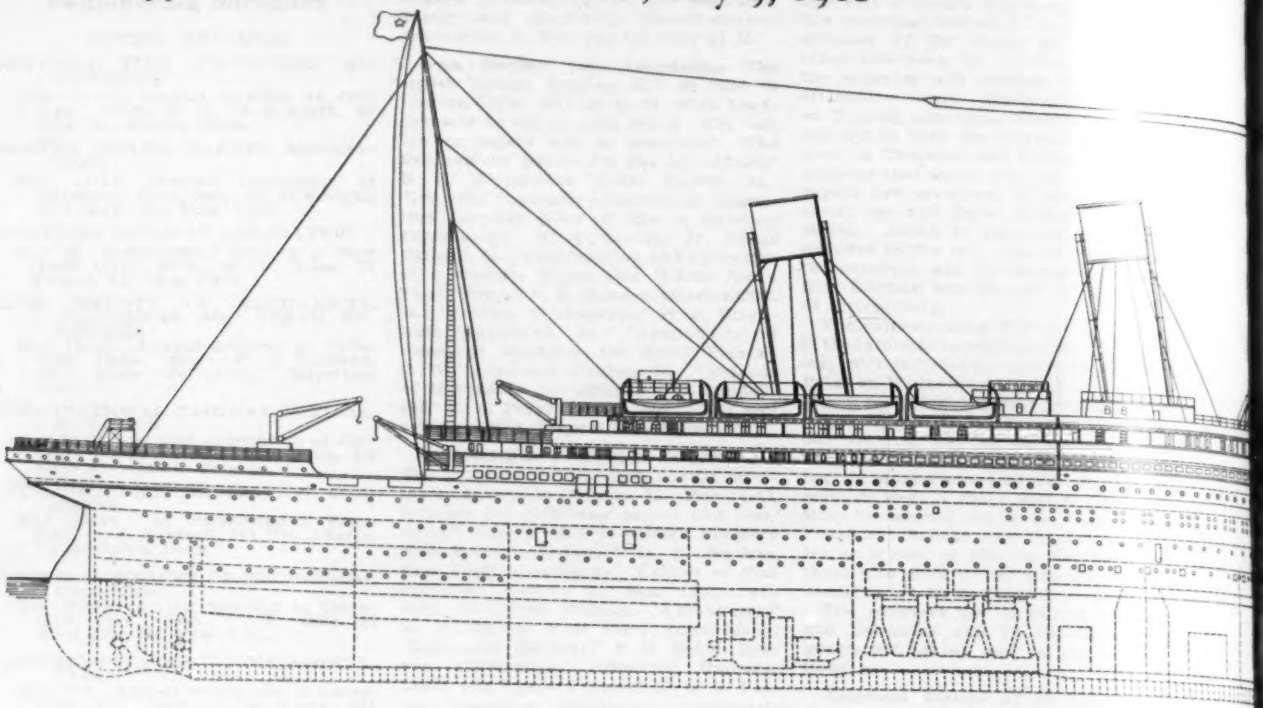
American Society of Mechanical Engineers—A tentative program of papers for the spring meeting, at Cleveland, Ohio, May 28-31, has been published by the society as follows: May 29—"A New Analysis of the Cylinder Performance of the Reciprocating Engine," J. Paul Clayton; "Equipment of Modern Flour Mills on a Gradual Reduction System," John F. Harrison and W. W. Nichols; "Design and Mechanical Features of the California Gold Dredge," Robert E. Cranston. Gas power section—"Problems in Natural Gas Engineering," Thomas R. Weymouth.

May 30—"New Processes for Chilling Cast Iron," Thomas D. West; "Strength of Steel Tubes, Pipes and Cylinders under Internal Fluid Pressure," Reid T. Stewart; "On the Control of Surges in Water Conduits," W. F. Durand; "Speed Regulation in Hydro-Electric Plants," William F. Uhl.

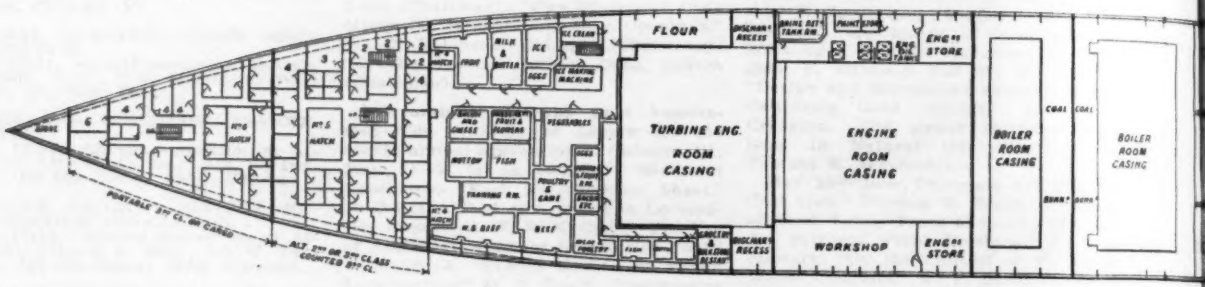
May 31—"The Present State of Development of Large Steam Turbines," A. G. Christie; "A Discussion of Certain Thermal Properties of Steam," G. A. Goodenough; "The Reduction in Temperature of Condensing Water Reservoirs Due to Cooling Effects of Air and Evaporation," W. B. Ruggles; "Results of Tests on the Discharge Capacity of Safety Valves," E. F. Miller and A. B. Carhart.

On Tuesday afternoon, May 28, an informal reception will be given at the home of Mr. and Mrs. Ambrose Swasey. The usual reunion and reception will be held Tuesday evening, at the Chamber of Commerce Hall. The professional sessions, at which the papers listed above will be presented, will be held in the forenoons. Wednesday, Thursday and Friday afternoons will be devoted to excursions and to other entertainment features. On Wednesday evening, May 29, Dr. Dayton C. Miller, of the Case School of Applied Science, will give an illustrated lecture on "Sound Waves; How to Photograph Them and What They Mean." On Thursday evening there will be a reception and dance at the Colonial Club.

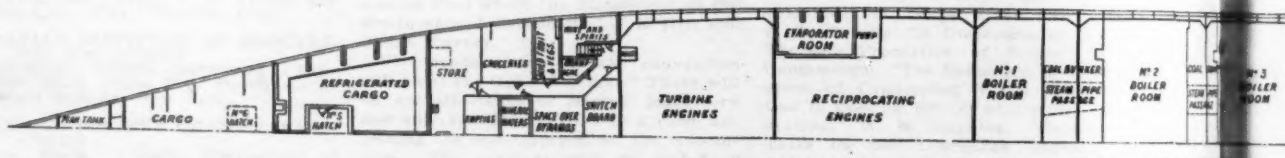




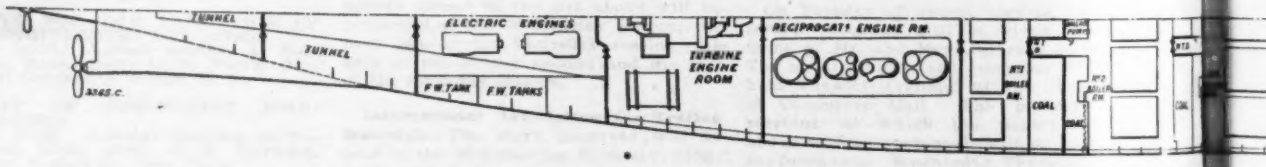
ELEVATION



PLAN OF LOWER DECK



HALF PLAN OF ORLOP DECK



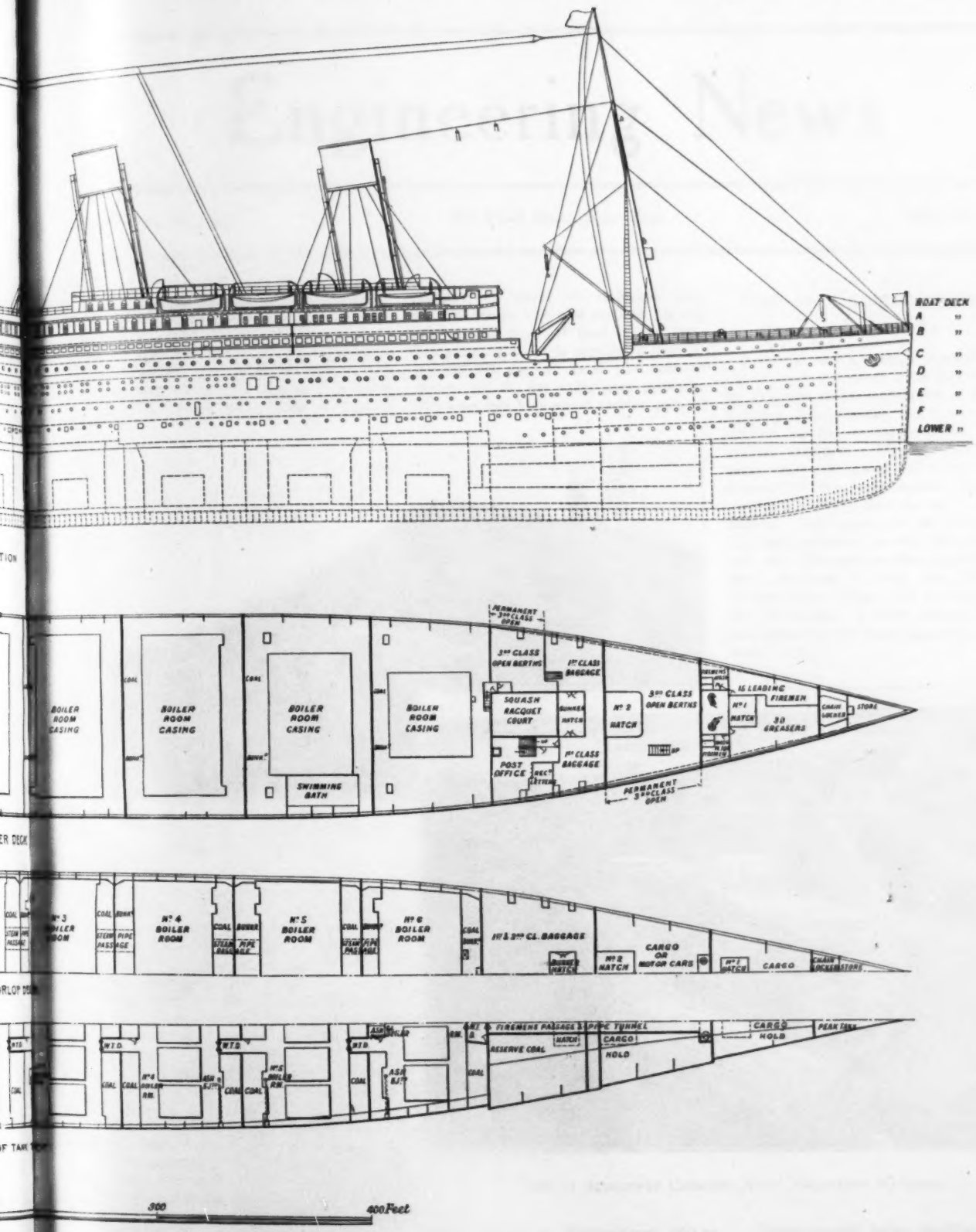
HALF PLAN OF TANK TOP

0 30 40 60 80 100 200

The White Star Steam

Sunk by collision with an iceberg off the N[illegible]dlan

Drawings reproduced from London "E[illegible]RING



STEAMER "TITANIC"

From the Newfoundland Banks, April 15, 1912

Based on "Engineering" of May 26, 1911